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Volume XXIII JANUARY, 1936 Number 1

SEWAGE DISPOSAL WORKS, BRISTOL, RHODE ISLAND.

BY RICHARD S. HOLMGREN, MEMBER *

Presented at a meeting of the Sanitary Section of the Boston Society of Civil Engineers, October 2, 1935)

THE placing in operation of the sewage treatment plant at Bristol, Rhode Island, completes one additional step in the effort of Rhode Island to bring the waters of Narragansett Bay back to their original beauty and purity.

In 1921 the State of Rhode Island, taking cognizance of the rapidly ncreasing pollution of the bay, passed legislation forming the Board of Purification of Waters, whose duties were to work with municipalities and industries discharging wastes into the bay, in order to arrive at means or stopping this pollution. In 1930 Bristol began construction of sewage disposal works, to stop discharging raw sewage and industrial wastes into the bay.

The original sewerage system was built in 1911, under the direction of Samuel M. Gray, consulting engineer, of Providence, and has at present a total length of about eighteen and one-half miles of sewers. It provided or discharge of sewage through three outfalls, — a 10-inch cast iron outlet at Oliver Street, a 12-inch outlet at Constitution Street, and a 6-inch outlet at the foot of Wood Street crossing Walkers Island. There were, in addition to these, several private outfalls from the mills discharging wastes directly into the harbor.

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* Chief Engineer of New Hampshire Water Resources Board, Concord, New Hampshire.

Location of Treatment Plant

The determination of a proper location for the treatment works presented some difficulties. The location of the present 16-inch outfall near the southern end of the town, discharging into deep water nearest the harbor entrance, indicated that this was the logical place at which the sewage should be discharged. The shores at this point are, however, lined with large and expensive residences, and it was therefore not considered advisable to locate the plant near the water's edge, but inland about 600 feet east of Wood Street, in an isolated spot, ideally wooded.

In 1931 the town meeting authorized an issue of bonds in an amount not greater than $225,000 for the purpose of constructing intercepting sewers and treatment works.

The plan as finally worked out called for a trunk sewer along Hope Street from Bradford Street to Wood Street, intercepting a large portion of the sewage originally flowing to the Constitution Street outfall and the present Walkers Island outfall, bringing the combined flows to the main pumping station on Wood Street near Hope Street, from which the collected sewage would be pumped to the treatment works.

A small pumping station located near the Constitution Street outfall pumps the remaining sewage, together with the industrial wastes to be discharged at this point, through a force main into the Hope Street intercepter. Another pumping station, located near the Oliver Street outfall, pumps this sewage through a force main along Oliver and Hope streets to the Hope Street intercepter at Bradford Street.

Pumping Stations

The Oliver Street and Constitution Street stations are quite similar, each containing two vertical units, with the pumps located in dry wells below the normal level of sewage, driven by motors at about street level. Sewage enters tangentially to adjacent circular wet wells with pump suction pipes leading from sumps at the center, resulting in a self-cleaning chamber. The Constitution Street units are each capable of delivering 400 gallons per minute against a head of 21 feet with a 5 horsepower squirrel-cage motor at a speed of 800 revolutions per minute. The Oliver Street units have each a capacity of 800 gallons per minute against a head of 24 feet. They are powered by 10 horsepower motors operating at 860 revolutions per minute. Each station is equipped with automatic controls actuated by floats, with provision for hand operation and for alternating the leading motor.

Sewage is diverted from the old outfalls to the pumping stations through control manholes located on the present outfalls. These are equipped with emergency overflows capable of discharging sewage in case of unexpected station shutdowns, and with shear gates for by-passing the stations in case of repairs.

The main station has three vertical pumping units, each with a capacity of 1,200 gallons per minute against a head of 29 feet, directly connected to 15 horsepower slip-ring motors operating at a speed of 690 revolutions per minute. The units are controlled by a float-operated switch so arranged that the speed of each unit and number of units operating can be varied to allow an increment change in sewage delivered to the treatment works of 600 gallons per minute.

Pumps are supplied by the Morris Machine Company, Baldwinsville, New York; motors by the Ideal Electric and Manufacturing Company of Mansfield, Ohio; and controls by the Sundh Electric Company of Cleveland, Ohio.

At the main station a Builders Iron Foundry Venturi meter is installed, with a Type Y register capable of indicating, recording and totalizing the amount of sewage pumped to the treatment works. No meters are installed at the two smaller stations, but recording ammeters are mounted on the switchboard, providing a record of operation, from which record the pumpage can be estimated.

INTERCEPTING SEWER

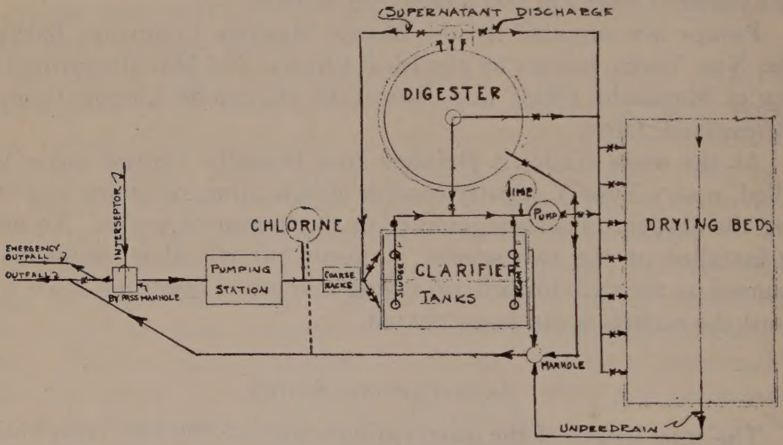
The total length of the intercepting sewer is 5,450 feet, lying entirely in Hope Street from Bradford Street to Wood Street. From Bradford Street to Summer Street it consists of an 18-inch reinforced concrete pipe, with a 24-inch reinforced pipe used for the remaining distance to Wood Street. Here it enters a control manhole provided with an emergency overflow and 20-inch shear gate connecting with the present Walkers Island outfall, making possible the by-passing of the treatment plant. From this manhole a 24-inch vitrified clay pipe about 450 feet long carries the sewage to the main pumping station, located on Wood Street, from which it is pumped to the treatment works through a 20-inch cast iron force main 780 feet long.

TREATMENT WORKS

Because of the large amount of dilution afforded in Bristol Harbor, coarse screening plain sedimentation and chlorination of the effluent, with separate sludge digestion, were adopted as the processes which would result in a satisfactory condition in Bristol Harbor. Control of the plant is centered in a control house, to which the sewage is pumped and started through the plant, and in which is centered the apparatus controlling the operation of the plant.

Control House

The main floor of the control house is divided into three rooms, — one containing the sewage racks, chlorine tanks with hoisting equipment and scales, and lime feeder; the second being equipped as a laboratory; and the third as a lavatory. A large plate-glass window is installed in the wall between the laboratory and the rear room, permitting the operator if in either room to see into the other. The chlorinator, manufactured by the Pardee Engineering Company, is mounted on a panel, which panel is also built into this division wall, with the gauges and controls in the laboratory and the working parts in the rack room. In similar fashion the chlorine scales are built with the platform in the rack room and the balance arm in the laboratory. The scales have 6,000 pounds capacity, with a 76-inch by 54-inch platform flush with the floor, built by Howe Scale Company of Rutland, Vermont. A hoist and trolley is provided, sufficient in capacity, with rail extending outdoors, to permit the unloading of the ton chlorine containers from a truck and placing them on the scales.



BRISTOL TREATMENT PLANT
FLOW DIAGRAM

FIG. 1

The rack-raking device is of the chain type, manufactured by the hain Belt Company of Chicago. The rack consists of 3/8-inch by inch copper bearing steel bars with a clear opening between bars of inch, and a channel area 3 feet wide and 3 feet deep. The entire quipment, including rack, rake and refuse container, is contained ithin a No. 14 galvanized sheet steel housing.

The lime feeder is of the dry feeder type, manufactured by the Inter-ational Filter Company, with a capacity of 0.10 to 36.0 pounds of ydrated lime per hour. The dry lime is deposited into a solution pot om which it runs by gravity into the suction side of the sludge pump.

The lavatory, in addition to the usual toilet facilities, is provided ith a shower bath and lockers.

In the basement is located a Rex single plunger sludge pump manu-actured by the Chain Belt Company, equipped with multi-speed motor nd adjustable crank for varying the plunger stroke. By combining the wo speeds with the various plunger strokes, six capacities can be ob-ained, varying from 22½ gallons per minute to 75 gallons per minute. Cast iron sludge withdrawal pipes 6 inches in diameter, from the hopper f the clarifying tank, are carried into the basement and valved before ntering the suction header of the pump. Similar pipes are brought into he basement from the scum hopper and bottom of digester and also valved before entering the header. The pump can therefore draw from ny clarifying tank hopper or from the digester or from the scum hopper. The pipe from the lime-feeding machine discharges into the suction pipe of the pump, so that lime can be fed into the pump when working and ntimately mixed with the sludge being pumped.

The discharge of the pump is so arranged that pump discharge can be to the digester, to the drying beds, or into the effluent line of the clarifier. By this means unwatering the clarifying tanks can be easily accomplished. The valve controlling the discharge to the plant effluent line is sealed by the State Purification Board, and is to be used only in emergencies, when the seal will be broken and a report made to the State Board.

In the basement is also located a heater burning gas from the di-gester, equipped with a circulating pump to force hot water from this boiler through coils in the digester. Operation is automatically controlled in order to keep the digester temperature at the best temperature for digestion. The gas burner is provided with all the usual safeguards to assure safe operation. A waste gas burner is also provided to burn all surplus gas not needed for heating. Gas meters are placed on the lines to the heater and to the waste gas burner, providing means of determin-ing the total gas wasted as well as that used.

CLARIFIERS

After passing through the rack in the control house the sewage flows by gravity to the clarifier tanks. Duplicate pipes have been installed leading from the screen chamber to each tank, in order to balance the flow between the two tanks. Stop log grooves have been provided at the entrance to each pipe, permitting the removal of one tank from operation without closing down the plant if repairs are necessary.

The tanks have an inside dimension each of 75 feet by 14 feet, with an average depth of 11 feet 6 inches and a normal sewage depth of 10 feet. At the influent end nearest the control house the floor of each tank is dropped 3 feet to form two hoppers.

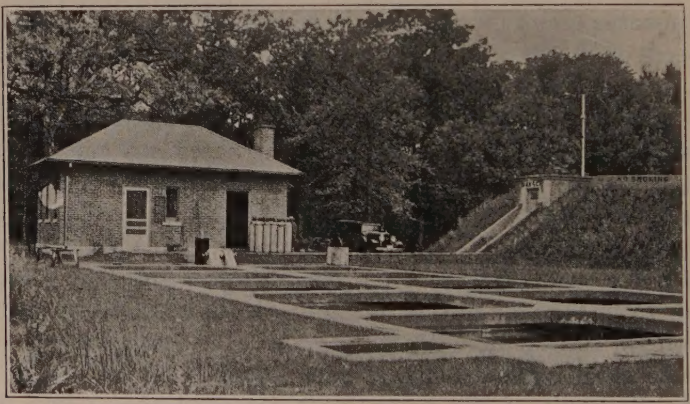


FIG. 2. — TREATMENT WORKS FROM SOUTHWEST CORNER OF CLARIFIER

The scraping equipment was furnished by the Chain Belt Company, and consists of wooden flights attached to chains which are drawn along the bottom, scraping the settled sludge into the hoppers, whence it is drawn by the sludge pump and pumped to the digester. On the return trip the flights are partly submerged and carry the surface scum to the effluent end of the tank, where it is easily scraped by hand into a scum trough, from which it flows by gravity into a scum hopper built adjacent to the tanks. This scum hopper is in turn connected to the suction end of the sludge pump, so that the scum can be pumped either to the digester or to the sludge beds.

Effluent from the clarifier passes over an adjustable weir into an ffluent channel discharging into an 18-inch vitrified clay pipe leading o a manhole near the control house. A 4-inch cast iron pipe is run from his manhole to the basement of the control house, providing a means of unning a rubber hose from the chlorinator to this manhole, so that the ffluent can be chlorinated if this is later found desirable. At this manhole the sewage enters an 18-inch reinforced concrete pipe line 1,500 feet ong joining the land end of the present 16-inch outfall, and is discharged at the entrance to the harbor in 20 feet of water.

A control manhole is provided where the concrete line joins the cast iron outfall, arranged so that when the pumping station is by-passed a valve can be opened and the sewage passed directly into the outfall. This valve is necessary because under normal operation the outfall line will be under pressure.

In case of an accidental shutdown of the pumping station a 12-inch cast iron relief line is provided running to low-water line, which will care or sewage until the valve into the main outfall is opened.

DIGESTER

The digester is a circular tank of concrete 34 feet inside diameter with an open top and a concrete bottom sloping to a 24-inch circular sump in the middle. The depth of the tank is 20 feet at the sides and 3 feet deeper at the center. The tank is equipped with a Pacific Flush-Tank Cover floating on the sludge surface, equipped for gas collection, with a total travel of 7½ feet. The elevation of the tank is such that sludge can be withdrawn from the tank to the drying beds by gravity with the cover in its lowest position. Supernatant can be withdrawn at three different vertical points, making withdrawal possible at the best point, with flow by gravity to the influent channel.

A valve chamber is located on the side of the digester nearest the control house, with an entrance from the side. Here are located the valves for withdrawing supernatant, which normally is returned to the influent line of the clarifier, but can be easily changed to flow directly to the drying beds. In this chamber are also located valves by which discharge from sludge pump can be sent directly to the drying beds. By this means scum can be pumped to the drying beds, and the lower portion of the digester, which will not flow by gravity, can be pumped to the beds.

A return pipe line running from the hopper in the bottom of the digester to the suction header of the sludge pump is provided.

DRYING BEDS

The sludge drying bed is 140 feet wide by 98 feet long divided into 7 beds 20 feet wide. Underdrains of 4-inch vitrified clay, laid open jointed crossway of the beds, drain into an 8-inch vitrified clay pipe with closed joints, which carries the drainage from the beds to the effluent channel of the clarifying tanks. The piping carrying the sludge from the digester to the beds is all 6-inch cast iron. The 4-inch underdrains are so laid that they can be easily flushed out from either end in case sand accumulates in them, with provision for rodding if this is necessary. The 8-inch closed drain is provided with manholes at each end of the beds for the removal of sand.

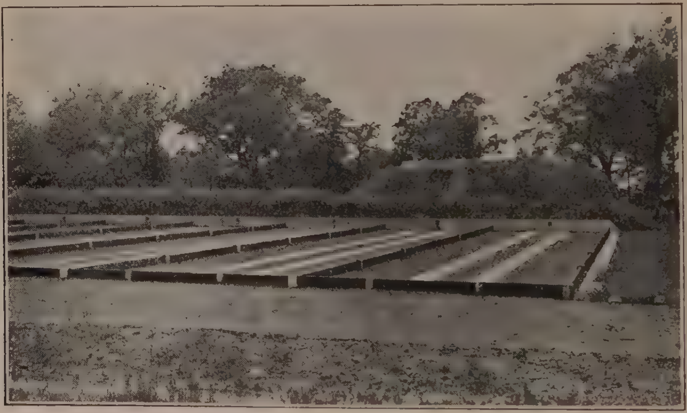


FIG. 3. — DRYING BEDS

The bed is composed of three layers of gravel, the lower layer of material 1 inch to 2 inches in diameter, surrounding all underdrains, with a maximum thickness of 6 inches. This is covered by a 3-inch layer of gravel ¼ inch to 1 inch in diameter, and this in turn by a 2-inch layer of gravel 1/16 inch to ¼ inch in diameter. After the gravel is placed and properly leveled and smoothed, the entire bed is covered with a 4-inch layer of sand.

Placed directly on the top gravel layer, and separated therefrom by building paper, are two concrete slabs running longitudinally along each bed, each 24 inches wide placed 36 inches apart. These slabs provide a driveway which will permit a truck to be backed along the bed as the dried sludge is removed.

The bed is surrounded and divided into sections by low partition alls consisting of concrete posts placed about 16 feet on centers with inch by 12-inch treated planks between them.

Flow from the digester to the beds is through 6-inch cast iron pipes o a header running the full width of the beds, with 6-inch gate valves ontrolling the flow to each bed. As this header is connected to the sludge ump, any stoppage in piping from header to beds or to digester can be ened by placing pressure on this header.

Design Data

A sanitary survey of the town was made to determine the distribuon of population contributing to each of the old outfalls. A survey of dustrial wastes to be treated was also made. Gaugings of the discharge each outfall were impossible, due to the backing up of the tide. It was erefore necessary to determine the flow in the main feeders to each outll and from these quantities estimate the probable flow. From the asic data thus collected the following basis of design was adopted:

otal population (1930 census)	12,117
ewer connections	1,530
onnected population (estimated)	11,000
stimated domestic sewage flows:	
Day, 7 A.M. to 7 P.M. (million gallons)	1.3
Day, average rate (million gallons daily)	2.5
24-hour average (million gallons daily)	1.9
Maximum rate (million gallons daily)	3.4
tercepter capacity based on 1970 population estimate	20,000
ump capacity based on 1940 population estimate	15,000
larifier:	
Total volume (gallons)	158,000
Detention period with average day rate (hours)	1½
igester, with controlled temperature 80° to 82° F.:	
Total volume (cubic feet)	16,500
Per capita (12,000 population) (cubic feet)	1.37
rying beds — open:	
Total area (square feet)	13,500
Per capita (11,000 population) (square feet)	1.25

ANALYSES OF SEWAGE

Domestic Sewage

Chemical and biochemical analyses of the domestic sewage show it to be a rather weak and fresh sewage. This was indicated by the following results:

	Bristol	Common Values
Ratio of fixed suspended solids (per cent)	28	20 to 30
Biochemical oxygen demand (5 days) parts per million	71	100 to 150
Oxygen demand per capita (pound/day)	.092	.12 to .17

Industrial Wastes

The only industrial waste of any amount produced in the town is that of the Collins Aikman woolen mill, consisting of wool-scouring and dye-house wastes. The wool-scouring waste is produced in two machines, with bowl capacities of 7,000 gallons and 5,350 gallons, respectively, which when operating two 8-hour shifts daily produce about 40,000 gallons per day of waste. On the basis of weak Bristol sewage, this industrial waste has a population equivalent of about 6,000 people. Based on a more normal sewage, the population equivalent would be about 5,000 people.

The dye-house waste, consisting mainly of salts and spent dyes, amounts to about 10,000 gallons per day.

COSTS

The project has been constructed under three contracts.

Contract I: —	
Intercepter and pumping station force mains	$43,90
Contract II: —	
Pumping equipment and controls	13,90
Contract III: —	
Pumping stations and treatment works	79,00
Total construction cost	$136,80

OTHER COSTS

and, rights of way, etc.	$10,000
ngineering and inspection	30,000
egal administration and overhead	5,000
iscellaneous	5,000
terest during construction	3,000
Total cost	$189,800

A breakdown of the construction cost is as follows:

TREATMENT WORKS

larifier	$11,240	=$7.10 per 100 gal.
igester	10,050	=$6.10 per 100 cu. ft.
rying beds	5,630	=$0.41 per sq. ft.
ontrol house, excluding equipment	9,670	
udge pump	700	=$7.20 per g.p.m. capacity
ack rake	1,430	
hlorinator	1,700	
ime feed machine	300	
onnecting piping	2,520	
Total	$43,240	=$17,300 per m.g.d. capacity = $3.93 per capita

PUMPING STATIONS

	Oliver Street	Constitution Street	Main Station
ntake piping	$990	$1,430	$2,520
tation structure	3,230	2,250	5,450
orce main	4,450	1,340	6,080
umping units and controls	3,600	2,800	7,000
	$12,270	$7,820	$21,050
apacity of pumping units	2 at 800 g.p.m.	2 at 400 g.p.m.	3 at 1,200 g.p.m.
ost per g.p.m. capacity	$7 70	$9 80	$5 80

PERSONNEL

The contractors engaged on the work were as follows:

Contract I. L. H. Callan Construction Company, Bristol, R. I.
Contract II. Starkweather Engineering Company, Incorporated, Boston, Mass.
Contract III. Simpson Brothers Corporation, Boston, Mass.

The Sewer Commission during the major part of the work consisted of Elisha Hibbert, chairman, William T. Dwyer, secretary, and Donat J. Gladue.

The design and supervision were by H. K. Barrows, consulting engineer, of Boston, Mass., with H. L. Scott as resident engineer and the writer as engineer in charge.

CHARACTERISTICS OF COHESIONLESS SOILS AFFECTING THE STABILITY OF SLOPES AND EARTH FILLS

BY ARTHUR CASAGRANDE, MEMBER *

Presented at a meeting of the Designers Section of the Boston Society of Civil Engineers, November 13, 1936)

INTRODUCTION

IT is not an exaggeration to state that throughout history and into he present day faulty designs have caused more loss of life and property n the field of earth and foundation engineering than in any other ranch of engineering. In this field the largest share of these losses ave been caused by failures of dams and dikes. Many times larger than he property losses due to failures has been the waste of money due to xcessive over-designing.

History is full of examples which show that experience accumulated uring many centuries will often lead to satisfactory solutions of problems long before they are properly understood by science. These solutions sometimes are so excellent that they can hardly be bettered by a modern, scientific approach. Therefore one may well ask why it is that ust in earth and foundation engineering our empirical knowledge, derived from the experience of many hundreds, even thousands, of years, as remained so unreliable.

On closer examination one finds, in the cases where experience has crystallized into empirical knowledge that can be relied upon, that certain conditions repeat themselves with little variation, or that the number of ariable factors is relatively small. In the case of soils we find just the contrary to be true. Soils exist in an almost infinite variety. In addition, he properties of an individual soil are so complex that the development of a scientific approach to the mechanics of soils could not take place ntil other sciences, particularly certain border fields between physics and physical chemistry, had reached a sufficiently advanced stage of evelopment.

I do not wish to convey the impression that I deny the value of experience in foundation engineering in general. But I do wish to differ-

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* Graduate School of Engineering, Harvard University, Cambridge, Mass.

entiate between experience which has proven its merit and which creates in the possessor a strong feeling that he can rely upon it, and such empirical methods which create a conscious or unconscious feeling of uncertainty, perhaps even a suspicion that one's design is, after all, not far from guesswork. Of all the empirical methods used in the field of earth and foundation engineering I consider those which have been used in the design of earth dams and dikes to be the least reliable. It is this subject, more particularly the *stability* of earth fills, consisting in part or entirely of *cohesionless* materials, which I wish to discuss in this paper.

The Meaning of the Term Stability

In mechanics a body is considered stable, or in stable equilibrium, when the forces acting upon it are in equilibrium and when small additional forces, for example, disturbances to which this body is commonly exposed, will not cause loss of equilibrium. If a slight change in the magnitude of the forces will result in an appreciable permanent displacement or movement of the body, then it is in an unstable equilibrium. Thus, a pencil standing on end, which loses its equilibrium when slightly disturbed, is, from a practical, or we may say, engineering standpoint, unstable. Naturally, such a meaning is not absolute. The limit between stable and unstable equilibrium is often subject to arbitrary definition; for example, if local conditions should make it practically impossible that a disturbance of sufficient intensity will act on the pencil to cause it to tumble, then we may say that the pencil is in a stable condition. However, if the table on which the pencil is standing is exposed to ordinary accidental disturbances, as tables usually are, then we should consider the pencil in an unstable condition.

A definitely stable condition is obtained by suspending the pencil from its upper end, because then it will always return to its original position. As another example of stability, let us consider a flexible board resting, like a beam, on two supports. In this condition the board is immune to ordinary disturbing factors. Even if I place a heavy weight upon it, causing considerable bending, its stability is not impaired; if anything, it is improved.

The pencil, when standing on end, has lost its equilibrium under a very slight disturbing force which, in itself, has not caused any noticeable deformation of the pencil. On the other hand, in the second example the beam deformed a great deal under the additional force, but its stability was not affected. Obviously, the stability of a body and its deformations due to the forces to which it is exposed are two independent

ects. Thus, the gradual settlement of buildings due to consolidation
underlying clay layers is not a stability problem, so far as the soil is
ncerned. If the building should collapse due to the settlements, it is
e building which has lost its stability, not the underlying masses of soil.
long as the stresses in the soil do not reach the shearing strength of the
aterial, it will undergo only deformations and volume changes of a sort
hich are comparable to the changes that structural materials, like wood
concrete, undergo when exposed to stresses which are smaller than the
ength of the material. However, if the external forces become so large
at the ultimate shearing strength of the soil will be exceeded in some
gion, then plastic flow or rupture will result and will initiate movement
a portion of the soil mass.

As long as the shearing stresses in the soil are considerably below its
timate shearing resistance, and as long as possible disturbances will not
iously affect the magnitude of the ultimate shearing resistance, the
ass of soil is in a stable condition, regardless of the magnitude of the
formations and volume changes which are caused by the stresses in the
il. However, if the external loads create stresses within the mass of
il that approach closely its ultimate shearing resistance, or if disturb-
ces result in a more or less sudden drop in the ultimate shearing
ength of the soil to a point where it approaches closely the existing
earing stresses due to the external forces, then it would be correct to
y that this mass of soil is not in a stable condition, or that it is unstable.
e various types of earth movements and slides, failures of earth dams,
ilures of foundations of dams* and other water-retaining structures due
"piping," etc., are examples of the loss of stability of a mass of soil.

Summarizing this discussion on stability, we may say that the in-
nctive conception which the layman has, when using this term, the
finitions used in mechanics, and also those conditions which we term
able and unstable when dealing with a mass of soil, are all inherently
e same. Stability of a mass of soil refers to the equilibrium of all ex-
rnal and internal forces with the resistance of the soil, including the
rce of gravity, seepage pressures, and any possible artificial disturb-
ces due to construction activities, etc., as well as the effects of earth-
akes. Stability does not refer to the amount of deformation which
ese forces produce, as long as the shearing resistance of the soil is not
ilized to its ultimate limit.

The stability of a mass of soil is not an individual property of the
aterial like the specific gravity, permeability, compressibility or angle

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* See "Application of Theories of Elasticity and Plasticity to Foundation Problems," by L. Jürgenson,
URNAL, Boston Society of Civil Engineers, July, 1934.

of internal friction, which can be measured on a sample of the soil and expressed by a single quantity. It is a combined effect of one or severa of such individual properties and of numerous other factors, particularly the character of the forces to which the soil mass may be exposed, its dimensions, various local conditions, and possibly other factors which are not sufficiently known; for example, the same cohesionless sand and gravel which forms a very stable and desirable foundation material fo buildings or roads will yield excessively under wheel loads if it is used as a road surfacing. It forms an unstable road surface because its shearing resistance under such conditions is easily exceeded. To "stabilize" this material for such purposes one has to increase its shearing resistance by an admixture of cohesive soil, tar or asphalt. This simple example should suffice to show that one should not speak of "stability" of a soil as if it were a definite soil constant, like its permeability or compressibility.

The Shearing Resistance of Cohesionless Soils

The shearing resistance of a cohesionless material is due entirely to the force with which the grains are pressed together, which, in general, is due to the weight of the overlying soil and the load of any superimposed structures. Therefore a deposit of dry sand has on its surface very little shearing resistance; yet only a few feet beneath the surface the weight of the overlying material mobilizes sufficient friction to make the sand very resistant to deformation. This can be convincingly demonstrated by placing dry sand in a rubber bag and evacuating the air. At first the bag feels like a soft cushion, but the moment some of the air is withdrawn the bag becomes as hard and rigid as a stone. The difference in air pressure between the outside and inside of the bag presses the sand grains together, duplicating the effect of the weight of a few feet of overlying soil, and the friction thus mobilized between the grains changes the soft cushion into a hard, solid block, conveying the impression that the grains were suddenly cemented together.

From this simple experiment we can see that the resistance of sand to deformation increases rapidly with the pressure between the grains By means of shearing tests it is possible to determine the relationshi between pressure and ultimate shearing resistance or shearing strength which can be expressed for cohesionless soils by Coulomb's law —

$$s = f \cdot p = p \cdot \tan \phi$$

in which s designates the shearing strength, p the pressure normal to th plane of shear, and f or $\tan \phi$ the coefficient of internal friction. Th angle ϕ is known as the angle of internal friction, and is identical with th liquity of the resultant force of the normal pressure and the ultimate earing resistance.

It is known that the angle of internal friction is dependent on the nsity of a sand and on its grain size, shape and uniformity. However, curate information on the magnitude of this angle for various types of hesionless soils is still surprisingly incomplete, and that in spite of the t that more research work has been expended in the course of time on is simplest of all mechanical soil properties than on all other mechanical operties.

Several factors, which apparently are of minor importance and have erefore been neglected, seriously influence the results of shearing tests, that today the numerous investigations which are available in pub- hed form disagree widely. The most important of these factors is the lume change associated with the deformation of soil, particularly of hesionless materials. An accurate observation of such volume changes of utmost importance, not only for the purpose of proper interpretation shearing tests, but also for the analysis of the stability of large masses cohesionless soils.

Volume Changes associated with the Deformation of Soils

If we observe carefully the volume changes of samples of sand dur- g shearing tests we find that *dense sand expands* and *very loose sand duces its volume.*

In a dense sand, as in Fig. 1*a*, the grains are so closely interlocked at deformation is not possible unless accompanied by a loosening up of e structure, as indicated by Fig. 1*b*. If dense sand is so confined that it nnot expand, then the shearing strength is determined by the resistance the grains to crushing, and therefore it acts essentially like a rigid one. This was observed fifty years ago by Osborne Reynolds, who monstrated it by filling a water-tight bag first with dry sand, in which ndition the bag could be deformed easily, and then replacing the air in e voids of the sand completely with water, after which the bag turned gid because the sand could no longer change its volume.

When the horizontal displacement and the volume change during a *earing test on dense sand* are plotted against the corresponding shearing ess (Figs. 1*e* and 1*f*), it is noticed that the shearing stress reaches a aximum S_D, — corresponding to the point *B* on the curve, — and if the formation is continued, the shearing stress drops again to a smaller lue, S_L, at which value it remains constant for all further displacement.*

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* This refers to the conditions in a large mass of sand. In a test the displacement is limited by the gn of the shearing apparatus. See "Recent Developments in Soil Testing Apparatus," by P. C. ledge, JOURNAL, Boston Society of Civil Engineers, October, 1935.

During this drop in shearing stress the sand continues to expand, as shown in Fig. 1*f*, curve *E–G*, finally reaching a critical density at which continuous deformation is possible at the constant shearing stress S_L. This critical density corresponds for very coarse, well-graded sand and for gravel approximately to the loose state of the material. For medium and finer grained sands it lies between the loosest and densest state. In addition, it depends to a large extent on the uniformity of the material. The more uniform a soil the lower the critical density.

The above also furnishes the explanation why most sands in their loose state have a tendency to reduce their volume when subjected to a shearing test under constant normal pressure. The shearing stress simply increases until it reaches the shearing strength S_L, and if the displacement is continued beyond this point the resistance remains unchanged. Obviously, the volume of the sand in this state must correspond to the critical density which we had finally reached when performing a test on the same material in the dense state. Therefore the curves representing the volume changes during shearing tests on material in the dense and the loose state must meet at the critical density when the stationary condition is established.

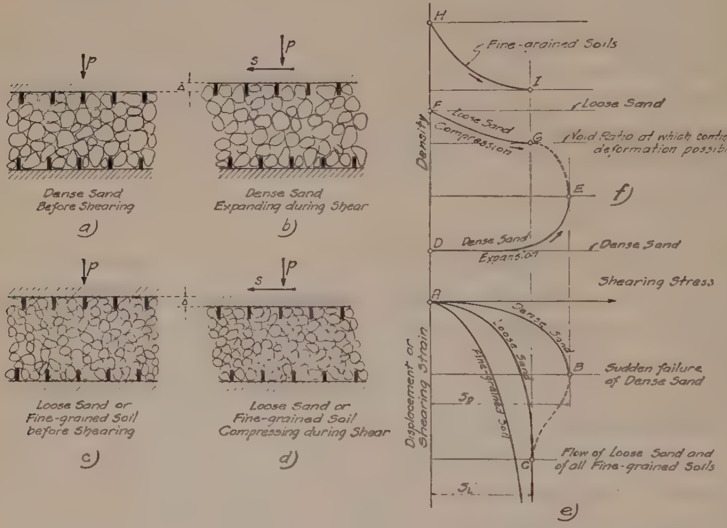

FIG. 1. — EFFECTS OF SHEARING ON THE VOLUME OF SOILS

For very fine-grained soils, like rock flour, inorganic silts, and clays, e volume usually decreases during a shearing test under constant nor-al stress, regardless whether the material was, at the start, in a loose ate or was compacted by vibrations, tamping, or static pressures of dinary intensity. In other words, for such materials volume decrease obtained much more efficiently by a combination of loading and de-rmation than by ordinary methods of compaction.

The most efficient method of reducing the volume of a cohesionless aterial is by means of vibrations. This is known not only to every one

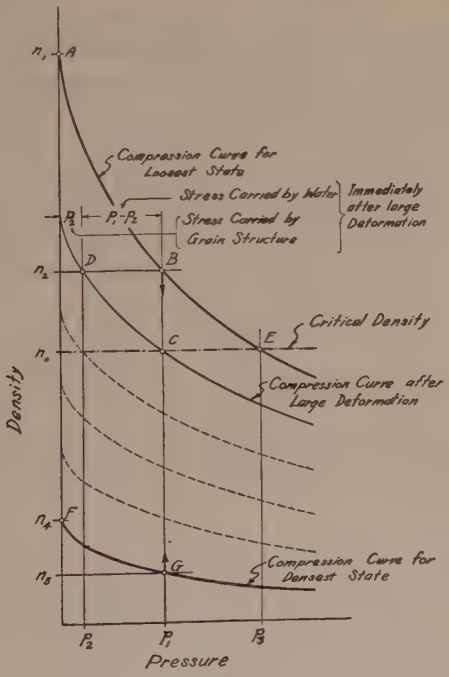

FIG. 2. — PRESSURE-DENSITY RELATIONSHIPS FOR SAND

ho has worked with such materials in the laboratory, but also to engi-eers who have observed the behavior of deposits of sand under the effect f pile-driving or vibrations caused by heavy machinery.

Static pressure is relatively ineffective in reducing the volume of a and; for example, it is not possible to change a loose sand into a dense and by static pressure alone. Professor Terzaghi has designated this roperty of retaining the essential characteristics of a loose structure, he "conservative structure" of sand.

By combining static pressure with simultaneous deformation, one can reduce the density of a loose sand very effectively. However, even this procedure will not make the structure of a loose sand as dense as is possible by means of vibrations.

We may compare the effectiveness of these three methods for compacting soil with the help of Fig. 2. The volume decrease under static pressure is determined by the compression curve which passes through the initial density. If the material was, at the start of compression, in its loosest state, n_1, then the static pressure p_1 will reduce the volume along curve *A–B*. If the material was at the start in its densest state, then the same pressure will reduce the volume along the much flatter curve *F–G*. The several compression curves, shown dotted, correspond to intermediate densities. Note that even high pressures, for example, 20 tons per square foot, will not reduce the volume of a loose sand sufficiently to come near that of the same sand in the dense state without load.

Suppose that a loose sand has been compressed under the pressure p_1 from *A* to *B*. If, in that condition, it is desired to reduce the volume further, this can be done by applying strong vibrations which will eventually reduce the volume to point *G*. Or we can apply a deformation to the soil, while keeping the load constant at p_1, which will reduce the volume until it reaches the critical density n_o at point *C*, in which state continuous deformation is possible without further change in density. However, no change in volume will take place if, at the beginning of deformation, the sand was at its critical density, and if it was below that density, deformation will result in expansion of the sand.

Volume Change during Deformation affecting Stability

As long as the sand can freely follow the tendency to change its volume during deformation, as is the case for dry sand, the stresses in a mass of sand are, at all times, fully transmitted from grain to grain. Therefore the shearing resistance, which is directly dependent on the normal stresses acting between the grains, is always fully active. Consequently, if the mass of sand was stable before deformation started, it will also be stable during the process of deformation.

If the voids in a mass of loose sand are completely filled with water, then the reduction in volume during deformation must be accompanied by an outflow of an equivalent quantity of water. If this volume decrease lags noticeably behind that which would take place if it were not hindered by the presence of water, the pressure between the grains will temporarily be transferred in part or entirely on to the water. It is this

ress in the water, known in soil mechanics as hydrodynamic stress (see
erzaghi, "Erdbaumechanik," 1925), which forces the excess water to flow
t. The time-lag of volume decrease is dependent on the permeability
the material, on the amount of reduction in volume, and especially on
e dimensions of the mass thus affected.

While the normal stresses in the soil are partially or fully carried by
e water, the pressures acting between the individual grains are reduced
a corresponding amount, since the total stress must remain equal to
e overlying loads. Simultaneously with this reduction, the frictional
sistance between the grains is reduced in the same proportion. The
nount of this reduction can be analyzed with the help of Fig. 2. Let
consider a volume element in the mass of sand in which the stress
nditions correspond to point B. This would indicate that the sand has
en deposited originally in a loose state and was subsequently com-
ressed by a static pressure equal to p_1. Now the mass of sand shall be
posed to horizontal forces which tend to deform it without, however,
anging the vertical pressures in the mass. That portion of the sand
hich is affected by the deformation will tend to reduce its volume. If
e deformation is sufficiently large, a new state of internal equilibrium
ill be established after the critical density n_o is reached in point C.
he compression curve through point C reflects the change in the struc-
re of the sand which was produced by the deformation.

If the quantity of water which could escape during deformation is
egligible, then the change in the structure without change in density
uses a drop in the pressure acting between the grains from p_1 to p_2,
e latter being determined by the intersection D between the abscissæ
rough B and the compression curve through C. The difference $(p_1 - p_2)$
now carried temporarily by the water and does not produce frictional
sistance, since the shearing strength of water is zero. The shearing
sistance in the zone of deformation, being proportional to the pressure
ctually transferred between the grains, is now reduced in the ratio
${}_2/p_1$. The pressure p_2 can be a small fraction of p_1, and may even be
qual to zero, so that temporarily the soil can lose a large portion of or
s entire shearing strength.

For very large pressures, which are not normally encountered in
oblems of earth and foundation engineering, even a loose sand may be
ompressed to its critical void ratio, as, for example, point E, and then
eformation will not cause any reduction in shearing resistance.

If the density at the beginning of the deformation was below the
itical density, as is always the case with dense sand, then deformation
a saturated mass will temporarily create tension in the water and a

corresponding increase in the pressure between the grains. Hence deformation of a mass of dense, saturated sand may result in an increase of the shearing resistance beyond the normal value. In other words, the mass seems to be bracing itself, to become temporarily more stable.

Comparison of Effects of Vibrations and Deformations

Loose sand can be efficiently compacted below the critical density only by means of vibrations. Sometimes it is not easy to distinguish between deformation and vibrations, and then one is forced to resort to an arbitrary definition. For our purpose we can utilize the volume change of sand for this differentiation. We may define deformation as any disturbance which, when continued long enough, will cause compression or expansion leading to the critical density. Vibrations may be defined as such oscillatory disturbances which, when continued long enough, will cause a decrease in volume below the critical density, and which, if applied with sufficient intensity, will compact the material into the densest state.

From the standpoint of stability it would appear at first that vibrations are more dangerous than a deformation. However, we must consider that large deformations can be produced in a very short time during which excess water may not be able to drain away, while vibrations must continue for long periods in order to produce an appreciable volume decrease. And during this time interval most of the excess water can escape without impairing the stability of the mass. Therefore it is my opinion, which would have to be substantiated by further investigations, that the stability of fills of loose, cohesionless sands, excepting very fine sands, may be affected more by quick deformations than by vibrations.

The most important natural disturbance to which a fill may be subjected is an earthquake. Whether an earthquake produces chiefly vibrations or deformations in the ground is, from the standpoint of the stability of saturated fills consisting of cohesionless soils, of great importance and should be investigated. It is my opinion that the vibrations themselves are not sufficiently effective and are not of sufficient duration to cause appreciable compaction.* However, the swaying movements which are amplified in large masses of sedimentary deposits, especially clay deposits, and also artificial deposits, like large earth dams, particularly when they rest on a clay foundation, produce quite substantial deformations in such masses, frequently leading to permanent displacements, shearing cracks, etc. And since these deformations take place in a few seconds, they may be accompanied by the formation of large hydrodynamic stresses and a corresponding reduction in shearing resistance which may lead to loss of stability.

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* This could be investigated by determining the density of natural deposits of cohesionless sands in regions subjected to frequent earthquakes.

It is my belief that only artificial vibrations continued during a considerable period, as, for example, pile-driving, or the application of special vibration machines, are capable of compacting cohesionless sands into a dense state. Foundation engineers are familiar with the fact that pile-driving frequently results in settlements of neighboring structures resting on deposits of loose sand. Pile-driving has been used for the purpose of compacting such deposits to improve their ability to carry loads without undue settlements. During the past few years methods of artificial vibration with special machines have been developed in Germany for the purpose of compacting artificial fills of sandy soils.

Effect of Disturbances on the Stability of Cohesionless Soils Illustrated by Means of Experiments

In the tank shown in Fig. 3 is deposited a fine quartz sand in a loose, saturated state, with free water standing on its surface, and a weight is placed on the surface. Then a stick is thrust into the sand, and suddenly the weight sinks below the surface. The slight but rapid deformation produced by the penetration of the stick results in a change of the sand structure and the formation of hydrodynamic stresses which quickly spread through the entire mass and so decrease the internal friction that the weight can no longer be supported.

This experiment can be made even more striking by letting water percolate in an upward direction through the sand, which changes it into an exceedingly loose state, and then draining away all water from the surface, so that capillary forces will be mobilized. In this state the surface can support a heavy weight without any noticeable subsidence. Upon driving the stick into the sand under these conditions the whole mass seems to liquefy suddenly and the weight disappears completely. In this case the liberation of water due to deformation produces, as a secondary effect, the disappearance of the capillary forces which helped materially in carrying the load.

Both experiments are imitations of conditions encountered in construction practice. The only difference is that natural sand deposits are hardly ever as loose as the sand in these experiments, and therefore the effects of any disturbance in nature are not so striking.

Of utmost importance is the question of the stability of sandy soils in dikes and dams. Contrary to the belief of many engineers that a mass of sand will always be stable if the slope is less than the angle of repose, there are many examples on record of embankments, dikes, etc., consisting of fine, saturated sands, being destroyed by flowing out of the entire mass as if it had been suddenly liquefied. Such flow slides have for the first time been thoroughly investigated and explained by Terzaghi. (See "Erdbaumechanik," pp. 344–352; also "Ingenieurgeologie," by Redlich-Terzaghi-Kampe.)

The essential difference in the stability of a dense and a loose sand, when subjected to severe lateral forces, like the waves of an earthquake, can be demonstrated by experiments such as those illustrated in Fig. 4. In two identical tanks, mounted on casters, model dam sections are built of ordinary beach sand, with slopes 1 on 2. One of the sections is built of sand in a very loose state, and the other of the same sand deposited in layers and tamped by hand into a dense state. On the downstream side the base is covered with a pervious filter blanket to keep the line of seepage away from the downstream face, which prevents sloughing and erosion.

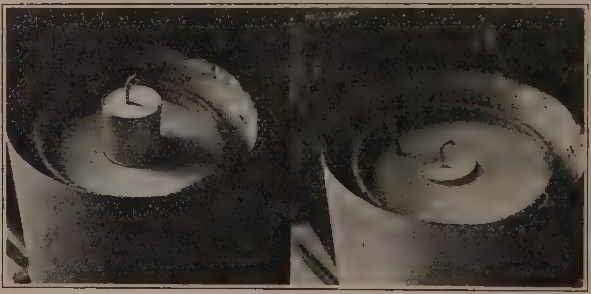

Fig. 3. — Loss of Stability in Fine-grained, Cohesionless, Saturated Soils

By shaking the tank containing the loose dam three or four times with a horizontal oscillation of about one inch, one causes the dam to flow out until the surface of the sand is nearly level. The other dam, consisting of dense sand, does not change its shape in the slightest, even if its tank is shaken much more violently and more frequently than the other tank.

After our preceding discussion a further explanation of these experiments is not necessary. However, it remains to analyze the possibility that one or the other factor is reduced or exaggerated in these experiments in its relative importance, as is the case in most model experiments. The four principal factors which determine the extent of reduction in shearing resistance are the amount of deformation, the intensity of volume decrease with deformation, the permeability, and the dimensions of the mass. If only the dimensions of the mass are reduced and the other factors kept constant, then the model will display a much smaller drop

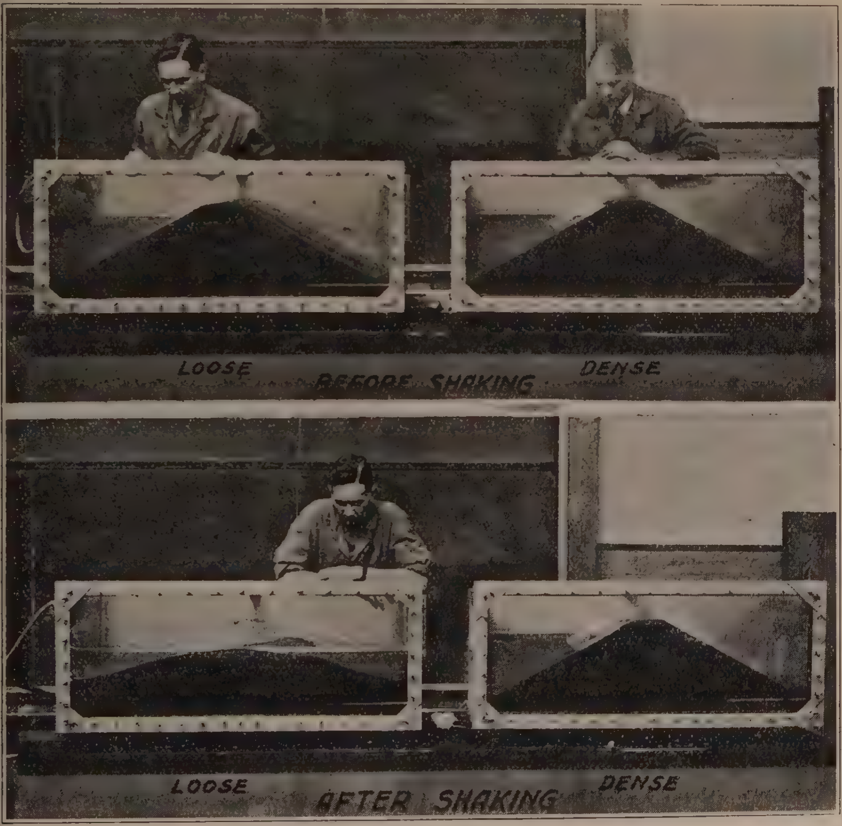

Fig. 4. — Effect of Horizontal Oscillations on Model Dam Sections Built of Sand

in shearing resistance and consequently a much larger stability than the prototype in nature. A measure for this difference is, first, the time required for the excess water to drain out, and second, the kinetic energy of the mass of soil accumulated during the motion for equal intensity of deformation. The time required for drainage increases, for the same soil, with the square of the height of the dam. The kinetic energy increases in direct proportion to the mass, that is, the square of the height, and furthermore, it increases with the square of the velocity. Since the average velocity is either independent of the height, for the same total displacement, or proportional to the height for equal angular displacement, it follows that the driving force increases at least with the fourth and possibly with the sixth power of the height. However, the resisting force increases only in direct proportion with the height. Therefore we arrive at the conclusion that, with the same type of sand, the amount of reduction in stability increases with at least the third power of the height of the dam.

If we wish to reproduce a model correctly we should reduce the permeability, and therefore the grain size of the material of which the model is built, to a very small fraction of that in the prototype, as Terzaghi has already pointed out in his book "Erdbaumechanik," page 348. However, such reduction leads to other difficulties, especially the presence of cohesion, and also the doubts which would probably be raised by engineers against the validity of test results obtained with such a very different material. The only way to make the tests with ordinary sand, perhaps of the same type as that in the prototype, and still be able to show the effect of deformation on the stability of the structure, is by exaggerating some other factor, for example, the amount of volume decrease which will occur during deformation. This recourse was taken in the model tests shown in Fig. 4, by starting out with a model in a much looser state than would normally be found in the prototype.

An interesting conclusion can be drawn regarding the amount of volume decrease necessary to produce a flow slide. The larger a structure is the smaller is the necessary amount of volume reduction to endanger the stability. With the same type and density of sand the larger structure would require a smaller deformation, or at the same deformation the larger structure must be denser than the small structure, to be equally stable. In other words, for a very large dam a density only slightly larger than the critical density may result in a flow slide.

Such experiments as shown in Fig. 4, are to me especially interesting, not on account of the behavior of the loose material, which I had anticipated before making the first tests, but on account of the behavior of the dense material, which indicates to me that a well compacted, cohesionless, saturated material is foolproof even against such disturbances as only one among a hundred dams may actually experience. Yet dams should be made safe even against such eventualities, if it is economically possible to do so.

EXAMPLES OF EARTH MOVEMENTS CAUSED BY FORMATION OF HYDRODYNAMIC STRESSES DUE TO REDUCTION OF VOLUME

In addition to the examples described by Terzaghi in the books "Erdbaumechanik" and "Ingenieurgeologie," the following examples may be of interest:

Recently a hydraulic fill dam in Russia failed, due to a flow slide which was initiated by the blasting of a near-by cofferdam. Russian specialists in soil mechanics who investigated this failure arrived at the conclusion that the failure was probably due to the development of hydrodynamic stresses which were created by the shock. Obviously, a large portion of the shoulders of the dam, which have the purpose of providing stability, must have consisted of loosely deposited sand, which when disturbed had the tendency to reduce its volume. It is probable that in this case the disturbance had more the character of very strong vibrations rather than that of a deformation, although this question must remain open. Certainly, it cannot be denied that the density of a large portion of the dam was above the critical density, and that an earthquake might have led to a similar failure. Furthermore, it seems to me beyond doubt that a thorough compaction of the shoulders during construction would have prevented such a failure.

Many disastrous land slides in deposits of hard or stiff clay, of ample strength to prevent slides, as long as they exist in a homogeneous mass, have their cause in the formation of hydrodynamic stresses within layers of sand or rock flour which are contained in the clay. Some natural or artificial disturbance causes the tendency to reduce the volume, and due to the slow rate at which the excess water drains away, the overlying mass of clay slides out as if suddenly placed on roller bearings.

The lack of stability of very fine-grained, saturated materials must also be considered in the placing of spoil banks of chemical wastes, which often consist of extremely fine powders. Excepting for a dry surface crust, the voids of such materials are filled with water. The large capillary pressures render it seemingly very stable, capable of standing on high vertical banks. Yet disturbances of any sort, or the formation of shrinkage or tension cracks with subsequent infiltration of surface water, resulting in deformations of the mass, will result in local liquefication of the material which may spread quickly over large areas and lead to a slide. An example of such a slide, shown in Fig. 5, had disastrous consequences. In the background of this picture one can see still standing the undisturbed banks of the deposit, 100 feet high.

Of great importance is the stability of the foundation materials on which dams or weirs are resting. It is known from experience that fine-grained sands are more liable to cause "piping" than coarse-grained sand or gravel. Professor Terzaghi* has offered an explanation why, especially in fine-grained, stratified, sandy soils, the danger exists of underground erosion and local subsidences which may lead to piping.

In my opinion it is quite possible that even in a homogeneous material piping may occur if the density is above the critical density. A serious disturbance, for example, an earthquake, or possibly even large seepage pressures, may result in a tendency to reduce the volume due to an adjustment in the structure, reducing the shearing resistance to such a point that the whole mass can flow like a liquid under the effect of the seepage pressures. The tendency for this kind of piping is again dependent on the permeability, density and the dimensions of the soil mass. Fine-grained materials are much more apt to exist above the critical density, and, in addition, their lower permeability results in more time being required for adjustments to take place so that a movement, once it has started, will inevitably lead to piping. The reproduction of such conditions by model experiments is rendered difficult for the same reasons as discussed above for earth dams.

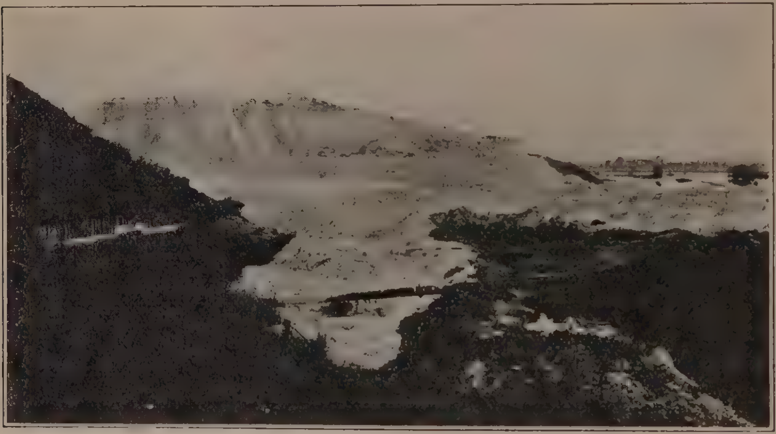

FIG. 5. — FLOW SLIDE IN CHEMICAL WASTES DEPOSIT

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* "Effect of Minor Geologic Details on the Safety of Dams," American Institute of Mining and Metallurgical Engineers, Technical Publication No. 215.

Whether, in such cases, indiscriminate increase in the coefficient of percolation over that required from the standpoint of the magnitude of the discharge gradient* represents a sufficient precaution is in my opinion an open question.

Methods for Compacting Cohesionless Soils

The question arises whether there are methods available which permit the compaction of cohesionless soils below the critical density. Natural deposits of loose sands can be effectively compacted by pile-driving. At various times it has been tried to create the vibrations by other means. These attempts were not very successful because the vibrations produced on the surface did not penetrate with sufficient intensity to greater depths. However, when compacting artificial fills, this latter objection does not hold, because one can build up the fill in layers, each of which can be compacted separately. During the past three years efficient vibration machines for such purpose have been developed in Germany for the construction of embankments of fine sands, in connection with the development of a system of superhighways. Investigations† have shown that, for *cohesionless soils*, such vibration machines are more effective than any of the other methods which were commonly used for compaction. Furthermore, it was found that the thickness of the layers in which the material can be sufficiently compacted can be greatly increased if compared with other methods. Layers up to eight feet in thickness are compacted satisfactorily.

It is interesting to note that simultaneously in the United States new and efficient methods have been developed for the compaction of *cohesive soils*, — for example, the sheeps-foot roller and the Proctor ‡ method for controlling the moisture content.

In my opinion it is only a matter of a few years until all fine and medium-grained cohesionless soils used in the construction of dams, dikes, etc., when exposed to saturation, will be compacted by means of vibration machinery, in order to eliminate any possibility of loss of stability due to unforeseen disturbances, and, last but not least, to permit greater economy by permitting the use of steeper slopes.

Whether stabilization by vibrations is necessary or not will often be a question that can only be decided by investigations. Particularly coarse sands and gravel may assume a density below the critical density without special measures for compaction. On the other hand, whenever large masses of fine, uniform sands are used in a fill, for example, for the pervious outer sections of large earth dams, our present knowledge is sufficient to indicate the necessity of stabilization by means of vibration.*

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* Transactions of American Society of Civil Engineers, 1935, pp. 1289–1294.

† "Verdichtung geschuetteter Daemme," by R. Mueller and A. Ramspeck, Die Strasse, No. 18, 1935.

‡ See "Fundamental Principles of Soil Compaction," by R. R. Proctor, Engineering News-Record, Vol. III, 1933.

If a portion of a dam consists of uniform, fine sand, and is subject to saturation, then its shearing resistance should not be taken into account in the design. This refers, for example, to the fine-grained, interior portions of the shoulders of a hydraulic-fill dam, when applying Professor Gilboy's stability analysis.†

REMARKS ON THE STABILITY OF COHESIVE SOILS

Most very fine-grained soils exist in their natural state above the critical density.‡

Thus the question suggests itself whether all these soils are liable to fail if subjected to disturbances. Here we must distinguish between such fine-grained soils which are not plastic and have only very little cohesion, for example, inorganic silts and rock flour, and plastic soils. The non-plastic, fine-grained soils, particularly if very uniform, are frequently of such character that all common methods for compaction, including vibrations, are not sufficient to compact the material below the critical density. There can be hardly any doubt that such materials are unsuitable in the construction of those structural sections of earth dams and dikes which are exposed to saturation. In foundation work such soils are also exceedingly treacherous, and it is certainly no coincidence that experienced foundation engineers have a sincere dislike for them.

However, as soon as we get into the plastic range, the effect of disturbances upon the stability of the soil is very much reduced. Vibrations have very little effect upon such soils and the effect of deformations is relatively small, due to their larger compressibility and their ability to undergo relatively large elastic deformations. The deformation required to reach the critical density of a plastic soil is many times larger than the deformations to which such soils may be exposed in earth and foundation engineering. Deformations of a magnitude which would cause, in a sand, the development of the critical density, will cause in a plastic soil only a slight drop in the compression curve, so that in Fig. 2 the quantity (p_1-p_2) is small in relation to p_1, and only a slight reduction in stability will develop.

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* After finishing this paper a publication appeared in Engineering News-Record of November 14, 1935, on "Man-made Earthquakes," by Franklin P. Ulrich, in which reference is made to investigations of the effect of earthquakes on masonry dams, and to the use of vibration machinery for various purposes, including the compaction of earth fills. The author would be very much interested to learn if earthquake experiments have also been conducted on models of earth dams.

† See "Mechanics of Hydraulic-Fill Dams," by G. Gilboy, JOURNAL, Boston Society of Civil Engineers, July, 1934.

‡ See "New Facts from the Research Laboratories," Engineering News-Record, September 5, 1935.

Recently in the Harvard Soil Mechanics Laboratory a cohesive and plastic soil was investigated, which was exceedingly well graded, containing large quantities of sand, and giving the impression that it was exceptionally well suited for the construction of a rolled earth dam for which it was intended. Subsequent investigations showed that the critical density of this soil corresponded to a higher water content than that at which it was intended to construct the dam. Hence, this material will be placed in the dam in a state below the critical density, and therefore it will be exceptionally stable. It is interesting to note that the good impression which this soil conveyed to experienced dam engineers was substantiated by laboratory tests. However, this example is not cited for the purpose of suggesting that only such cohesive soils should be used in dam construction for which the optimum water content* lies below the critical density. On the contrary, I wish to emphasize that we do not know enough about the behavior of plastic soils under shearing stress to be able to make any reliable statements.

CONCLUSIONS

1. Every cohesionless soil has a certain critical density, in which state it can undergo any amount of deformation or actual flow without volume change.

2. The density in the loose state of many cohesionless soils, particularly medium and fine, uniform sands, is considerably above their critical density. Such materials in their loose state tend to reduce their volume if exposed to continuous deformation. If the voids are filled with water and the water cannot escape as quickly as the deformation is produced, then a temporary transfer of load on to the water takes place, and the resulting reduction in friction impairs the stability of the mass, which can lead, in extreme cases, to a flow slide.

3. If a cohesionless soil is below the critical density, then it can stand any disturbance without danger of a flow slide. Whenever there is any tendency for the mass to deform, the water in the voids has a restraining influence.

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* See "Fundamental Principles of Soil Compaction," by R. R. Proctor, Engineering News-Record, Vol. III, 1933.

4. Many coarse-grained, and very well-graded mixtures of cohesionless soils are in their loose state approximately at the critical density. This fact, combined with their large permeability, renders them relatively stable against any disturbances, even in the loose state.

5. Cohesionless soils in a state above the critical density can be efficiently compacted, and thereby stabilized against any disturbances, by means of special vibration machinery.

WIRE ROPE

By J. F. Howe * and N. Carlson *

(Presented at a meeting of the Boston Society of Civil Engineers held on October 16, 1935)

It is impossible to condense into the space of a few minutes any great amount of information with regard to wire rope. However, an endeavor will be made to present a brief summary of the fundamentals which should be given consideration in any problem affecting the use of wire rope.

To begin with, wire rope is almost entirely a device for taking tensile loads. It is not practical to use wire rope in compression, and but rarely is it used in torsion. As a tensile mechanism, it must possess, first of all, strength, and there have been adopted by all rope manufacturers certain standard strengths which in their order, starting with the lowest, are as follows:

Iron grade, in which the wires have a tensile strength of 85,000 to 110,000 pounds per square inch.

Cast steel grade, in which the wires have a tensile strength of 180,000 to 200,000 pounds per square inch.

Mild plow steel grade, in which the wires have a tensile strength of 200,000 to 220,000 pounds per square inch.

Plow steel grade, in which the wires have a tensile strength of 220,000 to 240,000 pounds per square inch.

Improved plow steel or monitor grade, in which the wires have a tensile strength of 240,000 to 280,000 pounds per square inch.

These figures of tensile strength are approximate averages, because the diameter of wire entering into the rope affects the standard for tensile strength to a certain extent, the larger sizes of wire having the lower tensile strengths, and the smaller sizes having the higher tensile strengths in the same grade.

As will be noted from these figures, the tensile strength of rope steels varies from three to four times that of ordinary structural steel. This strength is secured by means of a suitable carbon and chemical content, one or more heat treatments known as patenting, and cold drawing from

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* American Steel and Wire Company, Worcester, Mass. This paper was presented at the meeting by Mr. Carlson.

the last heat treating stage. If strength were all that were required of rope wire, the problem would be simple, but as you well know, practically all wire ropes, with the exception of guy ropes and stationary ropes for suspension bridges, are subjected to additional requirements, including

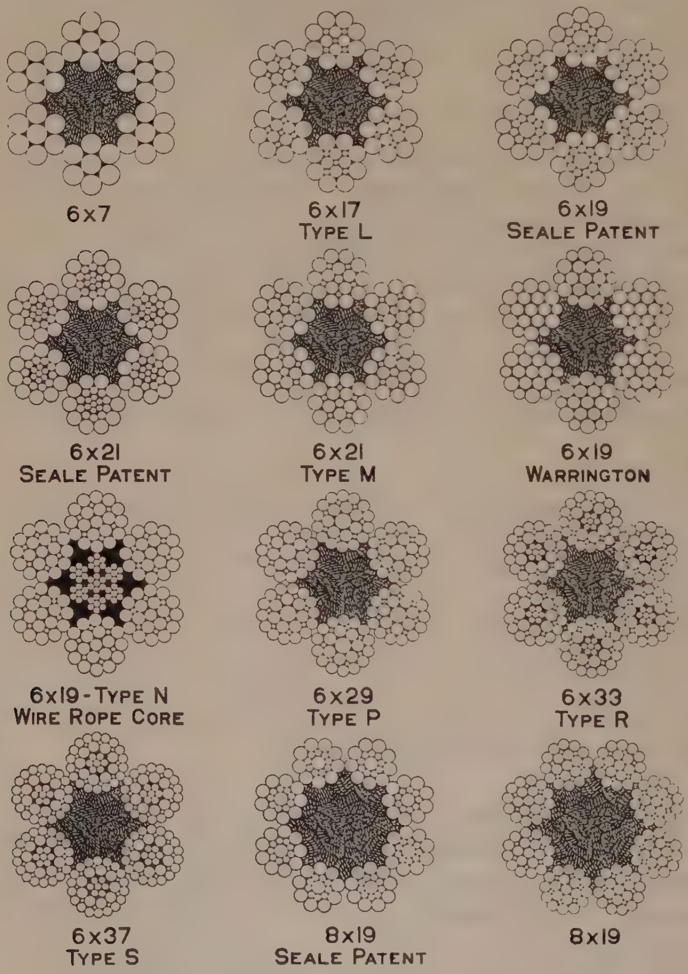
6x7
6x17 Type L
6x19 Seale Patent
6x21 Seale Patent
6x21 Type M
6x19 Warrington
6x19-Type N Wire Rope Core
6x29 Type P
6x33 Type R
6x37 Type S
8x19 Seale Patent
8x19

Fig. 1. — Cross Sections of Wire Rope

bending over sheaves and drums, external wear, corrosive action, shock loads, etc.

In order to meet these various conditions of service, it is therefore necessary to see that the wires possess not only tensile strength, but toughness, ductility, wearing qualities, and fatigue-resisting properties. While these properties are the result of selected chemical analyses, heat treatments, and wire-drawing practices, they are checked in the finished size of wire by tensile strength tests of wires to determine the grade, and then tested for torsion, bending and elongation to determine the ductility and fatigue-resisting properties. These tests are carefully performed on each end of each bundle of wire, thus insuring that the rope wire from which wire rope is made possesses suitable qualities to meet general service requirements.

The next point of importance to the rope user is the question of ability to resist bending conditions. This is accomplished by producing ropes of varying degrees of flexibility. It is obvious that ropes composed of few wires of large size will be comparatively stiff, while ropes composed of many wires of smaller size will be flexible. Between these two extremes lie the regular run of rope constructions, which in general are designated as 6 by 7, meaning six strands of seven wires; 6 by 19,—six strands of nineteen wires; 6 by 37, 8 by 19, etc. (Fig. 1.)

It is clear that to increase the number of wires in a rope construction will secure desired flexibility which can be added to the initial tensile strength requirement. But a third factor almost always enters into the problem, and that is, wearing quality. It is obvious that the larger the size of wire the greater its resistance to wear, and the smaller the size of wire the less its resistance to wear. Accordingly, instead of increasing the flexibility to an undue extent, we have to decrease it somewhat in order to obtain suitable wearing quality. However, ability to bend around a sheave or drum depends on flexibility; therefore if the wire sizes are increased too much it will result in rope that is too stiff when bent around sheaves and drums, which crack up quickly due to fatigue of the wires from the bending. Therefore it is essential that a balance be kept between the flexibility and the wearing qualities of the rope, so that the ultimate failure of the rope which results from a combination of these two will be postponed as long as possible. This is the goal of a wire rope manufacturer to which he devotes his utmost attention.

While tensile strength, wearing quality and flexibility represent three major factors in rope service, consideration must always be given to corrosion, shock loads, and various types of abuse due to conditions of service. Corrosion results from the absence of suitable lubrication, or the presence of poor atmospheric conditions. When corrosion takes place in a rope used for hoisting purposes, the entire rope structure is stiffened, due to lack of pliability induced by the oxidization of the steel of which the rope is composed. Under such a condition, a wire rope cannot give good service. It will usually fail prematurely, due to cracking of wires induced by the heavy internal friction. Suitable lubrication applied at regular intervals not only retards corrosion but reduces internal wear between the component parts of the rope.

Another condition which many ropes have to meet is that the diameter of the drum on the machine is not large enough to accommodate the rope in one layer, and as a result the rope winds several layers deep. This is known as overwind, and produces double or triple wear on the rope where it occurs. In addition, if loads are very heavy, overwinding frequently causes a distortion of the rope structure. This can only be partially prevented by the rope manufacturer in his design of the rope. It is a destructive condition, and one that exists in many places. It is, however, often unavoidable because of cramped conditions where rope has to be used.

It is desirable at all times to see that ropes wind smoothly and evenly on drums, but especially, if overwinding is present, be sure that the rope winds evenly as possible on the successive layers. Crisscross and irregular winding is very destructive to rope life, and may result in a dangerous condition of rope service.

Another feature that enters into wire rope service is the skill of the operator in handling the work which he has to do where machinery using wire rope is involved. A very material difference in service may ensue where ropes are used without the exercise of proper skill, and many times this is lost sight of in giving consideration to the service which successive wire ropes may give on the equipment. It pays to have the best possible operators on equipment using wire rope, because wire rope is one of the constant items of upkeep, and money saved there is well worth while.

It is possible in some cases for the rope manufacturer to help out where conditions are severe by designing a special rope, or changing the construction slightly, but where conditions are unusually hard, the manufacturer is frequently confronted with new difficulties in the attempt to provide a remedy for certain conditions.

The points that need to be borne in mind by a rope user or a designer are briefly as follows:

Make the sheaves and drums of as large size as practicable.
Eliminate overwinding on drums.
Do not arrange sheaves so as to produce reverse bending; reverse bending results in rapid fatigue.
See that the leads that the ropes make from drum to sheave, or sheave to sheave, are such that undue flange wear is prevented.

There have been many developments in the use and application of wire rope in the past score of years, and among items of particular interest o civil engineers is the use of prestretched rope for bridge work, guys on wireless towers, etc. A word in connection with this may not be out of place.

As ordinarily constructed, hemp center ropes constantly stretch hroughout their life, more at the outset and less at the finish. It is not desirable, and hardly practicable, to prestretch hemp center ropes, because the prestressing tension that would be required is much higher than he working stress that these ropes would be subjected to later. Furthermore, a stretch on a rope during its life is a safety provision, and insures good working conditions for the rope. For certain classes of work, how-ver, where wire center ropes can be employed, such as suspender ropes or suspension bridges, arch bridges, and for main ropes of suspension bridges that are constructed either of ropes or parallel strands composed f twisted wires, it has been found advantageous to prestretch the rope. For instance, such wire rope is ordinarily made with a wire center, and ossesses a modulus of about 13,500,000. After prestressing up to 50 per ent of its ultimate strength for a sufficient time, the modulus becomes 0,000,000. This increase in modulus is accomplished by taking out of he rope the structural stretch. Strands composed of 19 wires, 37 wires, nd even more where required may also be prestretched, and these will ive still higher values of modulus after prestretching, values running up o 25,000,000 or 26,000,000 being obtained without difficulty.

The introduction of prestressing thus gives a civil engineer a more eliable article to apply to bridge work, and this is one of the outstanding evelopments in the past six or seven years that has caused undoubtedly more improvement and reliability in wire rope or strands as applied to bridge work than anything that has been devised prior to that time. It as opened up a new field for suspension bridge work, making it possible o use prestressed strands of various sizes and numbers on such bridges s the Waldo-Hancock bridge in Maine.*

It is expected that there will be a wider use of prestretched rope and trands for bridge work in general. While it may be that the very largest uspension bridges, such as George Washington bridge in New York, an Francisco–Oakland Bay bridge, or the Golden Gate bridge, may still e constructed of parallel wires, it is believed to be practical to use parallel

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* Mr. R. M. Boynton, Assistant Engineer, Robinson & Steinman, Consulting Engineers, New York, resented a paper on the Waldo-Hancock bridge (Bucksport) before the Society on December 21, 1932.

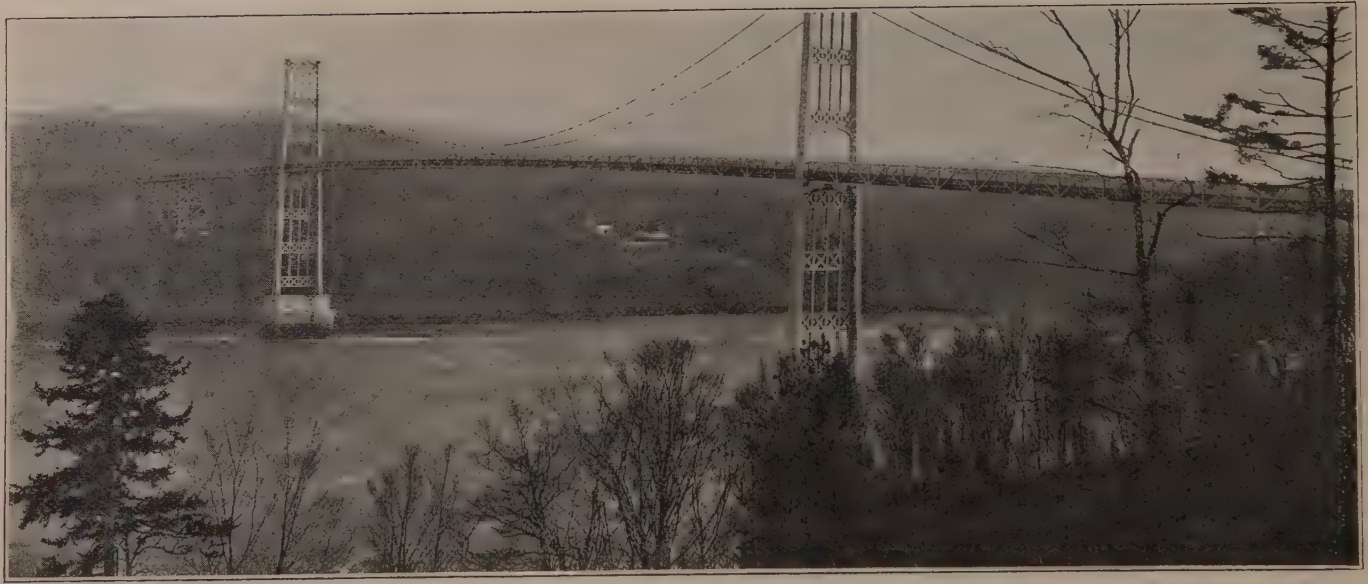

Fig. 2. — Waldo-Hancock Bridge, Bucksport, Maine
(Designed by Robinson & Steinman, Consulting Engineers, New York)

strands or parallel ropes for fairly large sizes of bridges, and thus speed up the erection very materially over present methods. We look forward to this, and hope that the trend of design will work more to the parallel strands and parallel ropes, and less to the parallel wire. The American Steel and Wire Company will be glad to have one of their engineers discuss these problems with any one desiring to obtain data on the construction of a suspension bridge in this manner.

RANDOM NOTES ON CABLE ENGINEERING

By Thomas F. Peterson *

(Presented at a meeting of the Boston Society of Civil Engineers held on October 16, 1935)

Presumably the appearance of an electrical engineer before a gathering of civil engineers requires some justification. Most certainly this is in order when the electrical engineer happens to specialize in cable engineering. Cable engineering, however, is unique because it requires a knowledge of many branches of science. Physics, chemistry and mechanical engineering are almost basic. Then, too, the cable engineer must be somewhat familiar with metallurgy, chemistry — yes, even botany and entomology. Since the cables in which we are interested serve primarily to carry electric current, it is assumed that electrical engineering must be the most important branch of science applicable thereto.

It would be possible to spend a full evening with these thoughts as a thesis. Attention could be given to such subjects as heat flow and its importance in rating cables, conduit systems, etc. Then there is the extremely complicated subject of vibration, particularly as applied to overhead transmission conductors. Also there might be some interest in corrosion or the characteristics of metals, such as copper, lead, steel, aluminum. These are the materials of which cables are made. They are combined in various fashions with rubber, paper, oil, resins, fibers, asphalts, lacquers, cotton, asbestos, jute, etc. It is comparatively easy to point out how dependent we are on a knowledge of chemistry of these materials.

A general discussion of cable engineering theory would be beyond the scope of my present opportunity. Therefore my remarks will be confined to those subjects and uses of cable with which you, as civil engineers, may be slightly familiar, but nevertheless those in which you are particularly interested.

At the outset one should dispel the more or less prevalent idea that "cable can be taken for granted." In the last ten years there have been fully fifty important ventures in which electric cable proved to be the life element standing between success or failure. In each case the particular cable had been viewed as something which could be obtained as,

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* Consulting Cable Engineer, American Steel and Wire Company, Worcester, Mass.

when and if wanted — something to be picked up from the nearest manufacturer. In most of these cases the time came when the customer almost begged us to assist in the solution of his problem. Successful solutions have been arrived at only after considerable trial and error, followed by ultimate research, testing and check-up of subsequent installations.

You are all, undoubtedly, interested in the progress being made in the building line. Under the government sponsorship, apartment house and small home building now reaches the stage where we can begin to refer to it as a boom. Numerous agencies are behind these movements. The government, manufacturers and utilities are strenuously pushing electrification of homes, farms and the like. Just how does a cable engineer fit into this picture? In the first place, the various standard methods of house wiring, such as Bx, wire in conduit, and the like, are being scrutinized with a view toward developing something cheaper. At the moment the utilities are actively sponsoring the so-called concentric wiring system in which one live conductor is insulated with rubber and then this is covered with copper wire stranded concentrically thereabout. This concentric application may be further protected with fibrous material, or with a wrapping of galvanized strip steel. Irrespective of what method is used, the outer conductor is uninsulated and always suitably grounded. Obviously, such cables weigh considerably less than those in vogue at the present time. If all the agencies, such as Underwriters' Laboratories, National Electric Manufacturers Association, inspection authorities, etc., ultimately approve this system of wiring, there is no question but that great savings will accrue, and the cost of wiring will be made more nearly commensurate with the lower rates and cheaper appliances now making their appearance.

Many of you have seen service entrances to buildings, but few, perhaps, have been aware of the fact that in spite of their high cost they were for the most part vulnerable and susceptible to current theft (politely called current diversion). Cable manufacturers have done a very fine piece of work in developing numerous types of cheap service entrance cables which now replace the old style conduit and wiring to meters. Incidentally, there is a trend toward outdoor metering, and these cables are easily adapted to such installation. As shown in Fig. 1 they are of a bare, neutral, concentric type. The outer covering of bare wires is grounded and serves effectively to prevent tampering. The cables are light and can be run directly from the pole to the meter without interruption or splicing.

Taking a rather big jump from the small dwelling to the Empire State Building naturally involves a corresponding large change in electric cable — call them service entrance cables, if you will. Ordinarily one is in the habit of thinking in terms of "horizontal" transmission and distribution of energy. However, when civil engineers conspire to build structures one third of a mile high, the cable engineers must face the problem of "vertical" transmission. Many of the large buildings in New York are supplied at 13,000 volts, and the cables carrying this voltage run vertically to as high as the 88th floor in the case of the Empire State Building. Just to make the problem more interesting, the architects limit us to very small spaces in which to install cable, and these are such that the cable must be supported from the top only. My own company has conducted expensive researches on so-called "vertical riser cables." These have been made up with rubber insulation, varnished cambric and paper, all of which have been non-leaded, but armored with galvanized steel wire. These were suspended from a smokestack and subjected to overloading and high voltage. The results of these investigations made it possible to build 13 and 27 KV cables for use in buildings when suspended from the top point only. Although there have been very few chances to apply our accumulated knowledge to installations in the newer buildings, the information has been found helpful in designing cables for vertical runs at power plants, such as that at Boulder Dam.

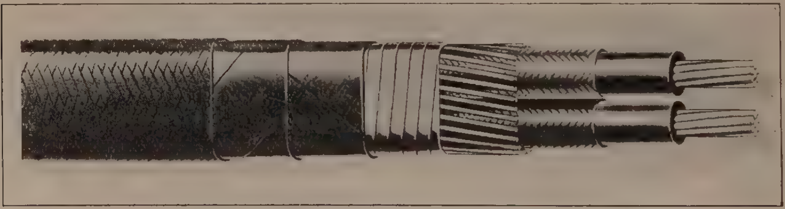

Fig. 1. — Concentric Armored Service Entrance Cable with Bare Neutral

The cable engineer's job is not finished when power is delivered to transformers, elevators, etc., at the top of skyscrapers. Indeed, some of the knottiest problems arise at lower voltages. Two rather classical cases will be cited, in which, as mentioned before, the cable was first considered to be just something to make the rest of the equipment work, but which turned out to be the very "heart of the system."

High buildings necessitated high speed, high rise elevators. Very little difficulty had been encountered with traveling cables (control, signal and lighting). However, when attempts were made to provide satisfactory cable, which incidentally had to be flameproof and colorcoded, elevator engineers realized that the successful operation of their equipment depended quite largely on cable design. Many elevators require 300 or 400 conductors which must travel at a speed of upwards of 1,400 feet a minute. Since space is limited and weights must be kept at a minimum, multiple conductor cables having as many as 42 conductors have been produced. Various designs, employing steel reinforcing strands, have been found necessary to withstand acceleration jerks and to resist fatigue, bending, etc. There are more problems in mechanics of materials and behavior of flexible structures encountered in design of these traveling cables than one would realize at first glance.

About six years ago an inventor developed a system of parking automobiles which, briefly, consisted in getting a given automobile on an elevator by using a small traveling carriage which engaged the rear-end housing of the automobile and then carried it on to the elevator. The car was carried to the proper floor and was then pushed out from the elevator (running between angle irons) by means of this same carriage, called a "parker." This device had two motors, one which propelled it, and the other used for driving the lift. A rubber jacketed cable supplied power to these motors and ran in and out from a reel on the bottom of the elevator. To the uninitiated, this service merely required a 6-conductor cord. Little did they realize that inability to get suitable cable almost spelled failure of the system. It so happened that cable had to be flexible and still have great tensile strength and abrasion resistance. These combinations do not always go together. Failure of a cable at a rush period might delay customers from thirty minutes to an hour. When these failures occurred as often as every week or so, the situation was considered serious. After very careful study of the problem we designed rubber-sheathed multiple conductor cables, which have given almost perfect service, at least over one or two year periods.

The government is to build a warship. The weight must be kept down. There are so many cables on an ordinary cruiser that the slightest saving in weight per foot is well worth striving for. Ordinarily we are engaged in building heavy power cables for utilities. In the last few years most of our attention has been given to more or less specialty demands, among which shipboard cables have been very prominent. These are, for the most part, flameproof and made up with asbestos and varnished cambric insulation, properly protected with flameproof coverings and final braids of aluminum wire. Many industries have co-operated in the development of materials required for these very special cables.

New England is a long distance from the oil fields, so that one may not fully appreciate some of the well-nigh miraculous things which are more or less commonplace in the oil industry. Many of the activities are dependent quite largely on special composite electrical-wire rope cables. (Fig. 2.) Several designs have been developed which perform numerous duties. One company is perforating oil wells which had become exhausted. A properly designed cable enables them to lower a chamber containing twelve or fifteen bullets, which, by means of fuses, may be fired one by one. The insulated electrical conductor in the core carries the firing current. The steel strands of the rope support the weight. A tremendous industry has been built up around this device, and the success of it depends quite largely on the cable. Others are using similar cable for electrical prospecting, operation of inclinometers, water-locating devices, seismographs, etc. Still another company has developed a 140-stage centrifugal pump which is lowered on the end of a pipe to the bottom of the well. The pump is driven by a 100 horsepower motor, 5⅜ inches in diameter and 24 feet long. A 3-conductor armored cable is fastened to the outside of the pipe at 30-foot intervals. Theoretically, this cable can carry no current. The operating temperatures exceed allowable temperatures even without current flowing. Nevertheless, the cable does drive the motor, and a very efficient unit is available for pumping from levels of 6,000 to 8,500 feet.

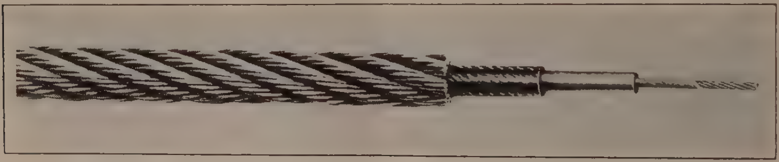

Fig. 2. — Steel Wire Rope with Insulated Conductor Core

No doubt it would be quite an exaggeration to claim that the building of one of the largest dams in the world was dependent on what one may facetiously call an electric cord. The United States government is building at Fort Peck, Montana, one of the largest earth-filled dams ever proposed. The dam will impound 19,500,000 acre-feet of water in a reservoir that will extend upstream approximately 180 miles. The main structure will have a height of 242 feet above the present bed of the Missouri River, a maximum base width of 2,875 feet, crest length of approximately 9,000 feet. The total length of the crest, however, will be 20,500 feet. To provide the 100,000,000 cubic yards of hydraulic fill that will be needed, four 28-inch, 12,500 horsepower electrically operated dredge units will pump water and silt from borrow beds immediately above and below the dam. Each unit consists of a suction dredge, a booster barge, and a land booster car with connecting pipe lines. After very careful studies the United States engineers chose electrically driven dredges. Power is transmitted about 300 miles from Great Falls, Montana, and then stepped down to 6,600 volts. Offhand it appears as though this voltage was somewhat low, considering the large blocks of power that had to be distributed over such large areas. The plan consisted in distributing by means of overhead lines to which portable cables were tapped. At 6,600 volts regulation problems were such as to require three or four 250,000 CM hollow conductor cables in multiple for each leg of the three-phase circuits. This resulted in some rather complicated overhead lines which had to encompass large areas, above and below the dam site. Spectacular as it might seem, it was somewhat dwarfed by comparison with the portable cables used from the line to the dredge. The accompanying photographs (Figs. 3 and 4) give some idea of the construction of the cable and its installation. Three 750,000 CM flexible stranded conductors are insulated with high grade 30 per cent rubber for 7,500 volts, and suitably shielded with copper braid. They are cabled together and subsequently

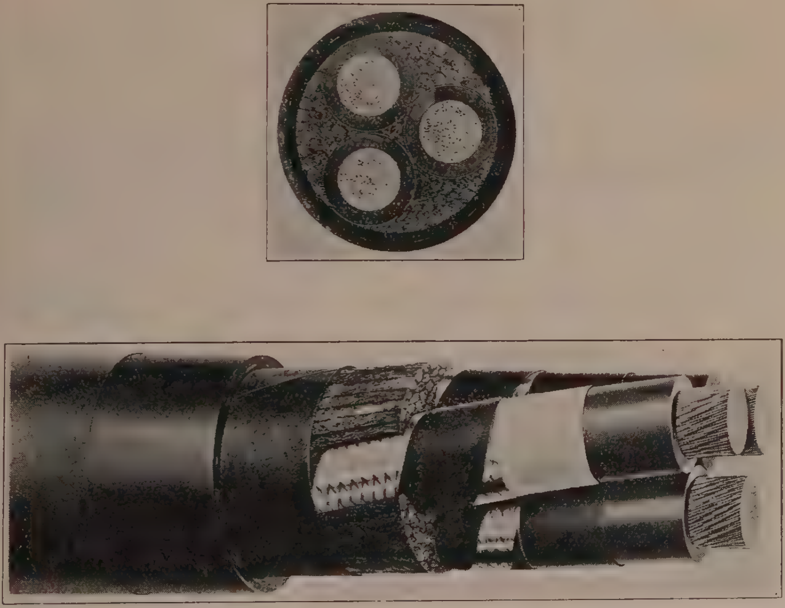

Fig. 3. — Portable Dredge Cable used at Fort Peck Dam, Montana

protected with a 60 per cent "tire stock" rubber jacket. Except for size, the construction is not unlike that of a sewing machine cord. However, this particular cord weighs 14 pounds to the foot, and was made up in 1,000-foot lengths. The outer jacket was vulcanized in a lead sheath which also weighed approximately 14 pounds to the foot. Difficulties encountered in the manufacture of this heavy cable seemed to be proportional to the cube of the weight per foot. Small cables are made without difficulty, but when such large cables are to be made, untold and unforeseen difficulties arise. Fortunately, all of them were successfully met, and there are now in operation about fourteen lengths of the cable in question. It is expected that by making repairs to damaged sheath, these cables will outlive the six-year period required for the job.

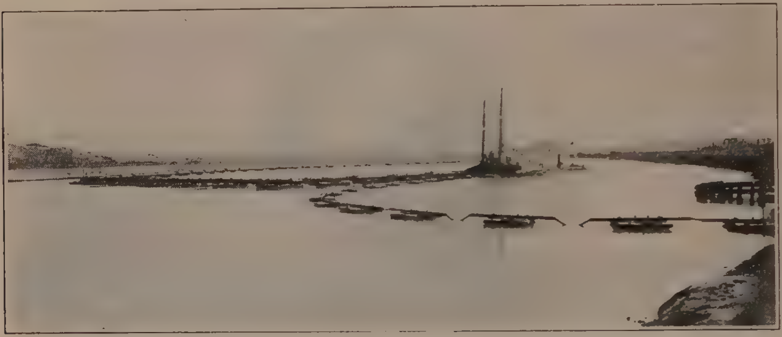

Fig. 4. — Dredge and Pontoons carrying Cable and Pipe Line at Fort Peck Dam, Montana

It is hoped that in skimming the field of cable engineering and choosing the high spots for review, I have touched upon one or two items of interest and have indicated that in the design or supervision of projects involving ordinary or special cable it is unsafe to "take cable for granted," or as something that "must be lived with."

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

NOVEMBER 21, 1935. — A regular meetng of the Boston Society of Civil Engineers was held this evening at Lorimer Hall, Tremont Temple, and was called to order by the President, John B. Babcock, d, at 7.15 o'clock.

This meeting was the annual joint meetng with the Student Chapters of the Amerian Society of Civil Engineers, including Student Chapters at Harvard University, Massachusetts Institute of Technology, Tufts and Rhode Island State College, which were well represented, and a large number from the Northeastern University Section of the Boston Society of Civil Engineers. About one hundred and sixty members and guests were present and one hundred and forty-five attended the buffet supper. During the supper, music was furnished by representatives of the musical clubs of Northeastern University.

The President announced the death of he following members: James W. Rollins, who died November 19, 1935, and had been a member since January 23, 1895; and Fayette F. Forbes, who died November 20, 1935, and had been a member ince May 21, 1879.

The President extended a welcome to he students and interestingly presented brief statement of the aims of the Society and the steps taken to interest junior members and students in the affairs of he Society.

The Secretary reported the election of the following new members:

Grade of Member: Morris B. Burstein,* Thomas C. Coleman, Jr., Joseph D. Guertin, William H. Kearns, John W. Lymberg,* Lester B. MacFarland.

Grade of Junior: Lloyd McLean Allen, Jr.

Grade of Student: George M. Anderson, Abraham Bickelman, Richard K. Butler, Kenneth M. Cline, Michael J. Colarusso, Felix J. Chludjinshi, John H. Corcoran, William G. Dyer, James Kline, Joseph C. McGann, William M. Mitchell, Lewis W. Pollard, John F. Shea, George T. Sorensen, William M. Stead, William A. Trowt, Edward J. Zaichuk.

The President then introduced the speaker of the evening, Dr. Harlan T. Stetson, Research Associate in Geophysics at Harvard University, who gave an extremely interesting talk on the "Sun's Effects on Human Affairs." The talk was illustrated by lantern slides. An informal question period followed the talk.

The meeting adjourned at 9 o'clock.

EVERETT N. HUTCHINS, *Secretary.*

Sanitary Section

DECEMBER 4, 1935. — A regular meeting of the Sanitary Section of the Boston Society of Civil Engineers was held December 4, 1935. The meeting was preceded by a dinner at Patten's, at which eighteen were present.

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* Transfer from Grade of Junior.

The meeting was called to order at 6.45 P.M., with thirty-eight present.

Mr. H. G. N. Hodgson, Division Engineer of Sewerage and Sewage disposal, Public Works Department, South Australia, presented a paper on Sewerage Disposal in Australia, with special reference to South Australia. His talk was illustrated by a large number of pictures showing the layout and results obtained in two plants near Adelaide in South Australia. His talk showed that these plants, recently built, have features which are advanced and up-to-date. The large number of questions asked pertaining to the operation and construction of these plants showed the great interest which was evidenced, by those present, in sewage work at this far-off place.

The meeting adjourned at 8 P.M.

RICHARD S. HOLMGREN, *Clerk.*

Designers Section

NOVEMBER 13, 1935. — The Designers Section joined with the Highway Section at a meeting held in Pierce Hall, Harvard University, Cambridge, as guests of the Harvard Graduate School of Engineering.

The meeting was opened by Chairman Brask at 7.05 P.M. in Room 110, and after welcoming the members and guests, the meeting was turned over to Prof. Arthur Casagrande.

Professor Casagrande then presented a talk on "Characteristics of Cohesionless Soils Affecting the Stability of Slopes and Earth Fills." The subject matter dealt particularly with the behavior of sands in various degrees of density, and with particular application to earth dams. The stability and permeability of loose and dense sand fills was illustrated by models of sand dikes, sustaining a hydrostatic head. These models were subjected to simulated earthquakes, and the results were strikingly apparent, with the collapse of the loose sand fill, and the continued stability of the dense sand fill.

Following Professor Casagrande, Mr. P. C. Rutledge, also of the Harvard Graduate School of Engineering, gave a talk describing the "Testing Equipment of the Harvard Soil Mechanics Laboratory." The problems encountered in the development of the apparatus was described, and the purpose and operation of the machines was explained.

After the speakers had concluded, the soil mechanics laboratories were open for inspection, and a great deal of interest was shown in the purposes and results obtained with the testing equipment.

The attendance was one hundred and twenty-five, including members of both Sections and guests. In addition to the courtesy of providing a meeting place, our hosts generously served refreshments, which were much appreciated.

ANTHONY S. COOMBS, *Clerk.*

DECEMBER 11, 1935. — This meeting of the section was called to order by Chairman Brask at 7.05 P.M. in Room 6–120, the Eastman Lecture Hall, Massachusetts Institute of Technology. The subject for the meeting was "Cape Cod Canal Research."

Mr. Brask introduced the first speaker, Lieut. John B. Elliott, United States Corps of Engineers, and assistant to Col. Kingman, District Engineer. Lieutenant Elliott gave a summary of the problems connected with the canal for which the research work is being conducted at the Institute. Lieutenant Elliott then turned the meeting over to Prof. Kenneth C. Reynolds of the Department of Civil and Sanitary Engineering, M. I. T., and under whose direction the model studies of the canal are being made. Professor Reynolds reviewed the history of the canal, and explained the necessity of the studies being made with scale models. The construction and general scheme of operation of the models were described, both of the complete model of the canal and the model of the east mooring basin. Both speakers illustrated their talks with slides.

After the speakers had concluded a visit was made to the laboratory, where the scale model of the canal was in operation. Professor Reynolds and his assistants explained the details of the model, with its very ingenious electrical controls. During the operation of the model, a predetermined tide cycle was automatically reproduced continuously, with automatic recordings and water levels at nine different points along the canal, corresponding with existing gauging stations. One complete tide cycle requires about ten minutes.

At present the model is being operated with profile and sections as it existed before widening, and being checked against actual gauge readings at known tides. After the model is calibrated and adjusted on this basis, the widened section representing about 540-foot bottom width and 32-foot depth will be reproduced, and studies made of the expected currents and tide levels.

This work is being done in co-operation with the United States Corps of Engineers.

About one hundred and twenty members and guests attended the lecture and inspection of the laboratory.

Anthony S. Coombs, *Clerk.*

January 8, 1936. — A meeting of the Designers Section was held this date at the Society Rooms, at 6.30 p.m.

The speaker was one of our section members, Albert Haertlein, Associate Professor of Civil Engineering at the Graduate School of Engineering of Harvard University.

The subject discussed was "The Design of Statically Indeterminate Frames."

Professor Haertlein reviewed briefly the fundamental formulas for obtaining the reactions in statically indeterminate arches and continuous trusses, and then gave a very instructive and interesting analysis of a method of shortening the very laborious process of the trial solution in which the areas of the members are ordinarily assumed. The talk demonstrated the fact that the areas of individual members are not absolutely essential for the trial solution, and was illustrated with blackboard sketches and graphs.

It is hoped that this valuable paper will be published in an early issue of the Journal.

The meeting adjourned at 8.10 p.m. with an attendance of thirty.

Anthony S. Coombs, *Clerk.*

Highway Section

November 13, 1935. — A joint meeting of the Designers and Highway Sections of the Boston Society of Civil Engineers was held on November 13, 1935, at 7 p.m., at Pierce Hall, Harvard University.

Prof. Arthur Casagrande of Harvard University spoke on "Characteristics of Cohesionless Soils Affecting the Stability of Slopes and Earth Fills." The principles controlling the stability of such soils were described and illustrated by experiments on models.

Following Professor Casagrande's talk, Mr. P. C. Rutledge described the special apparatus which has been developed at Harvard University for testing soils and determining the characteristics which must be known for scientific design.

Following this address the members and guests of the Society were invited to inspect the Soil Laboratories of the University.

The meeting adjourned at 8.30 p.m.

Attendance, one hundred and five.

A. J. Bone, *Clerk.*

Northeastern University Section

November 7, 1935. — A meeting of the Northeastern University Section of the Boston Society of Civil Engineers was held in Room 25–H on November 7, 1935. The meeting was opened by Vice-Chairman G. A. Smith at 8.15 p.m.

The meeting was in the form of a smoke talk. Professor Alvord, of the University, gave an interesting account of his recent trip across the country by auto.

There were about ten members present.

The meeting adjourned at 9.30 p.m.

L. S. Perry, *Clerk pro tem.*

December 13, 1935. — A regular meeting of the Northeastern University Section of the Boston Society of Civil Engineers was held in Room 18–H, December 13, 1935.

The meeting was called to order at 7 p.m., and the minutes of the October meeting were read and approved.

The speaker was Mr. Paul F. Howard, of Whitman Howard, whose subject was "Ground Water Supplies." Mr. Howard presented an instructive and interesting history of the methods of obtaining ground water. The talk was well illustrated with slides, and the advantages and disadvantages of each type, the problems encountered, and the recent trends were pointed out by Mr. Howard. A discussion period followed and refreshments were served.

The attendance was twenty.

The meeting adjourned at 8.30 P.M.

FRANCIS J. FLYNN, *Clerk.*

APPLICATIONS FOR MEMBERSHIP

[January 6, 1936]

THE By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission

CANNING, HUBERT MILES, North Abington, Mass. (Age 43, b. Bala, Pa.) Graduated Northampton, Mass., Grammar School; three years, Northampton High School; one year Williston Academy; tutored in civil engineering as applying to landscape architecture; also in landscape design over period of five years (1911–1916); July, 1910, to December, 1917, with Olmsted Brothers, Brookline, Mass., in planting department; advanced from clerk in purchasing to assistant in design and supervision of outside work; 1917–1920, United States Army Construction Division, as 1st lieutenant, Quartermasters Corps, in requirements branch, auditing and accounting; 1920–1933, owner and manager of Edw. J. Canning Company, Northampton, Mass., landscape construction, design and nursery work; designed and executed many private jobs in Connecticut Valley; landscape architect for United States Veterans Hospital at Northampton, 1923–1926; land subdivisions, grading and drainage plans on private jobs, 1920 to date; December, 1933, with Massachusetts Highway Accident Survey, becoming supervisor of District No. 2 and later chief traffic engineer of public safety studies; October, 1934, appointed Director of Highway Traffic Research, an E. R. A. project operating under Department of Public Safety, in highway safety work; at close of research work on this project became associated as a landscape architect under Emergency Planning Bureau, in charge of work in the five southeastern counties, co-operating in W. P. A. work with Mr. R. F. Cross, administrator, 4th District; at present, landscape architect with Emergency Planning and Research Bureau, Inc., 182 Tremont Street, Boston, Mass. Refers to *F. Tyler Carlton, J. D. Guertin, J. H. Harding, J. A. O'Hearn.*

CANTERBURY, MALCOLM PAGE, West Roxbury, Mass. (Age 38, b. East Weymouth, Mass.) Graduated Massachusetts Institute of Technology, 1921; during summer of 1921, assistant at Camp Technology; employed two months in 1921 as transitman on land surveying by A. S. Beale, Sandwich, Mass.; 1922–1923, as engineer on highway and construction work for D. J. Walsh, general contractors, Pittsfield, Mass.; 1924–1930, as field engineer and superintendent of construction for White Construction Company of New York, on industrial building; 1930 and 1931, in same capacity for Industrial Engineering Company of New York; 1930–1934, at intervals on land surveys and supervising construction for Yale Forest, Union, Conn.; July, 1934, to date, employed as engineer inspector on P. W. A. projects in Boston, Mass. Refers to *J. B. Babcock, 3d, J. W. Howard, E. Mirabelli, K. C. Reynolds, C. M. Spofford.*

CANZANELLI, ANDREW, Arlington, Mass. (Age 41, b. New York City.) Graduate of Massachusetts Institute of Technology, 1927; 1914–1915, Bureau of Survey, city of Philadelphia, Pa.; 1918, military service in Engineers Corps; 1919–1922, miscellaneous engineering positions; 1923 to date, with Metropolitan District Commission designing several bridges, sea walls, buildings, roadways, etc., and supervising construction on many sizeable projects. Refers to *J. B. Babcock, 3d, L. M. Hersum, E. Mirabelli, L. E. Moore.*

CHILDS, ELLIOT F., Waltham, Mass. (Age 27, b. Waltham, Mass.) Was graduated from Waltham (Mass.) High School in 1926; 1926–1927, attended Northeastern University, then transferred to Massachusetts Institute of Technology, graduating in 1931; studied civil engineering in both colleges, specializing in hydro-electric work at M. I. T.; at M. I. T. was elected to civil engineering fraternity, Chi Epsilon; as an undergraduate, spent two summers as an assistant instructor of surveying at M. I. T. summer school; from October, 1931, to May, 1932, was employed as an assistant on sewer project, Bristol, R. I.; since July, 1932, have been with engineering department of city of Waltham on street construction, sewerage, drainage, estimating, pumping station construction work, all of which include office and field work; September, 1934, to May, 1935, during the erection of bridge over the Charles River was field engineer. Refers to *J. B. Babcock, 3d, H. K. Barrows, W. M. Fife, J. R. Worcester, Thomas Worcester.*

DONAHOE, JOHN T., Brooklyn, N. Y. (Age 38, b. Boston, Mass.) Attended Franklin Union, Boston, Mass., 1914–1916; courses with University Extension, Massachusetts Board of Education, 1920–1922 and 1932; College of the City of New York, 1932–1934; April, 1913, to January, 1918, with F. E. Sherry, Boston, Mass.; January, 1918, to April, 1918, with Boston & Maine Railroad; April, 1918, to January, 1919, with Whitman & Howard, Boston; January to February, 1919, with C. H. Gannett, Boston; March, 1919, to January, 1921, with F. E. Sherry; February to March, 1921, with Whitman & Howard; March to June, 1921, with F. E. Sherry; September, 1921, to April, 1923, with H. F. Bryant, Brookline; April to August, 1923, with F. E. Sherry; August, 1923, to January, 1926, with Public Works Department, United States Naval Training Station, Newport, R. I.; January, 1926, to June, 1927, with Public Works Department, United States Navy Yard, Boston, Mass.; June, 1927, to June, 1928, assistant engineer with Rock Creek & Potomac Parkway Commission, Washington, D. C.; June, 1928, to September, 1935, design draftsman with Public Works Department, United States Navy Yard, Brooklyn, N. Y.; September, 1935, to present time, assistant engineer with Public Works Department, United States Navy Yard, Brooklyn, N. Y. Refers to *W. A. Bryant, Channing Howard, P. F. Howard, C. D. Thurber.*

EVANS, HOWARD TASKER, Wellesley, Mass. (Age 42, b. Haverhill, Mass.) Graduate, Massachusetts Institute of Technology, architectural engineering; 1916–1917, with S. M. I. Engineering Company (associated with Thompson & Lichtner Company, Boston) as structural designer, draftsman, laboratory worker and inspector, under Edward Smulski; 1917–1920, United States Army; honorable discharge, May, 1920, rank of captain, Coast Artillery Corps; 1920–1921, structural designer, with the Koppers Company, Pittsburgh, Pa.; 1922, automobile sales work; 1923–1932, and intermittently to date, with Stone & Webster Engineering Corporation, progressively, designer, squad boss, junior engineer and engineer; 1932, superintendent of construction and maintenance for John Donnelly & Sons; 1933, sales engineering, industrial heating and oil burning, Petroleum Heating and Power Company, Boston, and superintendent of construction of concrete shipways, Navy Yard, Boston; 1934, with United Engineers and Constructors, Philadelphia, engineer on a temporary job; 1935, acting public works commissioner for the city of Fitchburg, Mass., during leave of absence of the regular commissioner; at present, executive engineer, Navy Yard, Boston, in connection with the administration of a group of Federal W. P. A. projects. Refers to *L. S. Cowles*, *E. W. Huckins*, *C. E. Nichols*, *T. B. Parker*.

KENNEY, ALBERT A., Wellesley Hills, Mass. (Age 35, b. Brookline, Mass.) Graduate, Brookline High School, 1919, and Massachusetts Institute of Technology in 1923; after graduating from Technology had several temporary jobs with Weston & Sampson, engineers, and Aberthaw Construction Company, Boston, and Boston Senate Investigating Committee investigating Bureau of Internal Revenue at Washington, D. C., and Moore & Burns, engineers, at Clearwater, Fla.; have been with Chicago Bridge and Iron Works of Chicago, Ill., for the past ten years; the last six and one-half years have been manager of the Boston office, in charge of their contracting work in New England. Refers to *J. B. Babcock, 3d*, *H. K. Barrows*, *C. B. Breed*, *R. W. Horne*, *C. W. Sherman*, *C. M. Spofford*.

MARBLE, RALPH A., Fitchburg, Mass. (Age 53, b. Ashburnham, Mass.) Graduate of Cushing Academy, Ashburnham, Mass., 1902; three years at University of Vermont, civil engineering course (left on account of ill health), 1906; instrumentman and draftsman, with city engineer, Albuquerque, New Mexico, 1906; civil engineer and surveyor, Estancia, New Mexico (self-employed), 1907–1916; land surveying, mining and irrigation engineering, county surveyor, Torrance Company, New Mexico, 1912–1916; 1916–1924, general engineering practice (self-employed), Nutrioso, Ariz.; 1925–1930, employed as civil engineer by Parker, Bateman & Chase, Fitchburg, Mass.; 1930, purchased business of former employers and have continued in the practice of engineering to present time; resident engineer on Brooks Mill Dam, 1931–1932; chief engineer, Fort Devens Landing Field, 1933; member, Fitchburg Planning Board, 1934–1935; technical supervisor, Nashua River Valley Survey, 1935. Refers to *H. B. Allen*, *C. E. Pethybridge*, *H. M. Turner*, *A. D. Weston*.

MASON, AUSTIN B., Weston, Mass. (Age 49, b. Boston, Mass.) Harvard, A.B., 1908; Massachusetts Institute of Technology, B.S., 1910, in civil engineering; 1910–1913, with Stone & Webster Engineering Corporation, on two large water power plants in the West, surveying, chainman, transitman, chief of party, etc., and six months in drafting room; 1914, in engineering, associated with C. Hillsmith in Boston, chiefly designing and supervising installation of heating, ventilating and electrical work; 1916–1918, in the war, part of time doing some engineering on construction of aviation camps; 1919–1934, with the Massachusetts Mohair Plush Company of Boston, and from 1922 on, in position of treasurer; September, 1934, to date, student at Harvard Graduate Engineering School; now studying for doctor's degree in civil engineering; received degree of S.M. at Harvard in 1935. Refers to *A. Casagrande*, *G. M. Fair*, *A. Haertlein*, *E. N. Hutchins*, *H. M. Turner*.

PUTNAM, GERALD, Arlington, Mass. (Age 33, b. Clinton, Mass.) Graduate of Boston English High School, 1918; postgraduate course at English High, 1919; S.B. degree in civil engineering, Massachusetts Institute of Technology, 1923; July, 1923, to May, 1924, engineering department of New England Telephone and Telegraph Company, Boston, Mass.; May, 1924, to January, 1926, estimator and detailer of reinforced steel, Kalman Steel Company, Boston, Mass.; January, 1926, to May, 1928, draftsman and designer of steel and concrete construction, Stone & Webster, Inc., Boston, Mass.; May, 1928, to August, 1931, engineer in office and field on water supply project for city of Albany, N. Y., with Whitman, Requardt & Smith, Albany, N. Y.; October, 1933, to date, instructor in civil engineering department, M. I. T., Cambridge, Mass. Refers to *J. B. Babcock, 3d*, *H. K. Barrows*, *C. B. Breed*, *G. E. Russell*.

RUTLEDGE, PHILIP CASTEEN, Cambridge, Mass. (Age 29, b. Champaign, Ill.) September, 1923, to June, 1927, Harvard College, Cambridge, Mass., S.B., 1927; September, 1927, to June, 1928, Massachusetts Institute of Technology, undergraduate in architectural engineering; June, 1928, to May, 1929, Barney-Ahlers Construction Corporation, New York, N. Y.; transitman, chief of party, assistant superintendent on foundation and reinforced concrete construction jobs; June, 1929, to May, 1931, Stone & Webster Engineering Corporation, Boston, steel draftsman, steel detailer, detail checker, steel designer, design checker, assistant chief of structural squad, chief of structural squad in field office in Virginia; September, 1931, to January, 1932, United States Army, Field Artillery School, Fort Sill, Okla.; student officer, battery officers course; February, 1932, to June, 1933, M. I. T., graduate student in civil engineering, S.M., 1933; September, 1932, to June, 1933, assistant in soil mechanics, M. I. T.; September, 1933, to June, 1935, research assistant in soil mechanics, Graduate School of Engineering, Harvard University, also conducting research toward Sc.D. degree; instructor in soil mechanics, Graduate School of Engineering, Harvard University, Cambridge, Mass. Refers to *A. Casagrande, J. S. Crandall, G. Gilboy, A. Haertlein, H. A. Mohr.*

Ryan, John Michael, Boston, Mass. (Age 30, b. Philadelphia, Pa.) Graduated from Massachusetts Institute of Technology in 1928, with degree of B.S.; employed with Department of Public Works, Highways, Commonwealth of Massachusetts, since October 13, 1926, one year of which was part time; the work in the department included plotting and tracing highway plans, drawing up cross sections and profiles, estimating cost of construction, drawing up designs showing grade, alignment and drainage; and for five months assisting in field supervision of construction; now, junior civil engineer, highway section, Department of Public Works, Commonwealth of Massachusetts. Refers to *A. B. Appleton, G. A. Graves, A. E. Kleinert, F. H. Morris, R. J. Roche.*

Ryder, Lincoln Woodbury, Wollaston, Mass. (Age 24, b. Wollaston, Mass.) In June, 1933, graduated with degree of C.B., Massachusetts Institute of Technology; July to November, 1933, Property Management Corporation, Boston, assistant in real estate management; January to March, 1934, Massachusetts Traffic Accident Survey, Boston, draftsman; March to June, 1934, signalman, triangulation under direction of corps of engineers, Army Base, Boston; July to August, 1934, and November, 1934, to February, 1935, with Metcalf & Eddy, Boston, as assistant engineer on design of sewerage treatment plant; March to May, 1935, rodman, under city of Quincy engineering department; May to September, 1935, United States Army; resident officer on active duty, assistant district construction officer, Third C. C. C. District, Fort Devens. Refers to *J. B. Babcock, 3d, H. R. Barrows, C. B. Breed, S. T. Drew.*

Whiting, William Curtis, West Hanover, Mass. (Age 28, b. Boston, Mass.) Graduated in 1920 from Massachusetts Institute of Technology with degree of B.S. in civil engineering; from November, 1929, to date, with Eastern Underwriters Inspection Bureau as fire protection engineer. Refers to *J. B. Babcock, 3d, C. B. Breed, J. D. Mitch, C. M. Spofford.*

For Transfer from Grade of Junior

Everett, Albert Ellsworth, Auburndale, Mass. (Age 33, b. Medford, Mass.) Northeastern University, 1923, B.C.E.; 1935, B.S. in civil engineering; 1932, M.B.A.; employment prior to June, 1923, as a co-operative student at Northeastern University; October, 1919, to February, 1920, Woodfall & McClintock, rodman and instrumentman; February, 1920, to June, 1922, C. H. Gannett, civil engineer, instrumentman; June, 1922, to September, 1925, Boston & Albany Railroad, rodman, instrumentman and engineer in charge of track relocation and construction; September, 1925, to January, 1926, Mexican Petroleum Corporation, district engineer; January, 1926, to April, 1926, Factory Mutual Fire Insurance Companies, inspection department; April, 1926, to

September, 1926, T. A. Cox, engineers, chief draftsman; September, 1926, to April, 1927, Factory Mutual Fire Insurance Companies, inspection department; April, 1927, to date, Northeastern University, co-ordinator of co-operative work. Refers to *H. B. Alvord, C. O. Baird, E. W. Branch, C. S. Ell.*

Greenleaf, John Whittier, Jr., Brighton, Mass. (Age 27, b. Springfield, Mass.) Graduated from Northeastern University June, 1930; 1927–1928, rodman and transitman, engineer's office, town of Stratford, Conn.; 1928–1930, rodman, transitman, chief of party, engineering department, city of Waltham, Mass., being part of education at Northeastern University; since June, 1930, employed as assistant engineer in office of X. H. Goodnough, Inc., sanitary engineers, Boston, Mass. Refers to *H. B. Alvord, E. S. Averell, C. O. Baird, F. H. Kingsbury, A. D. Weston.*

Lowe, John Howard, Lynn, Mass. (Age 29, b. Lynn, Mass.) Was graduated from Lynn English High School, 1927; degree of B.C.E. from Northeastern University in 1932; September, 1928, to June, 1932, during school work periods, with Aspinwall & Lincoln, Boston; December, 1933, to present time, employed as senior engineer in Lynn, Mass., under C. W. A., E. R. A. and W. P. A. Refers to *H. B. Alvord, C. O. Baird, L. A. Chase, C. A. Wolfrum.*

Richardson, Charles S., East Lynn, Mass. (Age 27, b. Lynn, Mass.) Was graduated from Northeastern University with degree of B.C.E., in 1929; under co-operative working plan, one year, with Bradford & Weed, civil engineers, Lynn, Mass.; four months with Eastern Massachusetts Street Railway Company, and shorter periods with the city of Medford, Mr. Perkins (North Scituate), and Sawyer Construction Company; after graduation, in June, 1929, until December, 1930, with Boston office of Concrete Steel Company as designing detailer; December, 1933, to May, 1935, employed under Federal C. W. A. and E. R. A. programs, having transitman's rating; August, 1935, to date, with city of Lynn engineering department, as transitman. Refers to *H. L. Burton, T. R. Lyons, G. H. Melcher, L. T. Schofield.*

Soule, Ralph M., Braintree, Mass. (Age 29, b. Middleborough, Mass.) Graduated from Northeastern University, June, 1929. June to September, 1929, with H. L. White, Braintree, Mass., as chief of party and instrumentman on survey work; September, 1929, to November, 1930, field work and drafting, special studies on Boston Harbor investigation, Commonwealth of Massachusetts; November, 1930, to present time, with Commonwealth of Massachusetts, Department of Public Health, Division of Sanitary Engineering, on inspection, field work and drafting. Refers to *H. B. Alvord, T. R. Camp, F. L. Flood, F. H. Kingsbury, A. D. Weston.*

For Transfer from Grade of Student to Grade of Junior

Balmer, Robert Ray, Jr., Salem, Mass. (Age 22, b. Salem, Mass.) Was graduated from Northeastern University June, 1935, with B.S. in civil engineering; now working as a machine operator in the Danvers Bleachery until work is found in civil engineering field. Refers to *H. B. Alvord, C. O. Baird, C. S. Ell, A. E. Everett, Jr.*

Barron, Reginald Arnold, Branford, Conn. (Age, 23, b. Branford, Conn.) June, 1935, graduated from Northeastern University with degree B.S., in civil engineering. Refers to *H. B. Alvord, C. O. Baird, J. N. DeSerio, A. E. Everett, Jr.*

Butler, Walter Everett, New Bedford, Mass. (Age 25, b. New Bedford, Mass.) Was graduated from Northeastern University with B.S. degree — from five-year course — in civil engineering, in 1934; while at Northeastern University was with Frank M. Metcalf, civil engineer, for short period; November, 1930, to January, 1933, with city of Newton engineering department, in alternate five-week periods of co-operative work, as rodman and transitman; June 19, 1934, for nine weeks with United States Engineers on triangulation for fire control stations, New Bedford Harbor defence; October 2, 1934, to March 22, 1935, with E. R. A. as transitman and chief on various projects sponsored by building department, New Bedford, Mass.; from March 23, 1935, to date, junior engineer on E. R. A. planning work for city of New Bedford, Mass. Refers to *H. B. Alvord, C. O. Baird, A. E. Everett, Jr., A. E. Harding, G. H. Nye.*

Davis, Howard William, Lynn, Mass. (Age 24, b. Lynn, Mass.) December, 1930, to November, 1933, attended civil engineering course at Northeastern University; 1934, passed Massachusetts Extension Course in "Highway Construction and Maintenance;" now taking the "Wilson Structural Course" given by Wilson Engineering Corporation, Cambridge, Mass.; September, 1931, to April, 1932, rodman and transitman with the Lynn city engineering department; May, 1932, to October, 1932, private work developing one-half square mile of land into streets and house lots; May, 1933, to October, 1933, suburb land development and private engineering; November, 1933, to present time, chief of party and project director for C. W. A., E. R. A. and W. P. A., working under Lynn city engineering department, on development of 0-acre tract into a cemetery, including complete work in street, drain, water main design, and field work, also design of sewerage system to connect suburb with main system, and many sewer and drain designs and layouts; field work in construction of a highway; two complete designs for pedestrian underpasses; during the summer of 1935 was field engineer in charge of survey party on construction of the Wonderland dog track, Revere; at present, W. P. A. senior engineer, Lynn city engineering department, on the design and field work of Lynn's W. P. A. projects. Refers to *H. B. Alvord, C. O. Baird, C. S. Ell.*

Dearborn, Joseph M., Boston, Mass. (Age 23, b. Boston, Mass.) Graduate of Northeastern University, class of 1935, with degree of B.S., in civil engineering; served as student vice-chairman of the Northeastern University Section of the Boston Society of Civil Engineers, 1934–1935; while at Northeastern University did occasional work as a student field instructor in surveying; June to September, 1935, in hotel work of a non-engineering nature, with the exception of some maintenance work (Bridges Hotel Company, Biddeford, Me.); since September, 1935, and at present time, with L. L. Dearborn, government electrical contractor, as an assistant in estimating and figuring contracts, layout of conduit lines, etc., some design work of a minor nature, and installation — this work has been in connection with post offices, hospitals, etc., for the government in the New England district. Refers to *H. B. Alvord, C. O. Baird, C. S. Ell, A. E. Everett, Jr.*

Dunning, Richard Lewis, Brookline, Mass. (Age 25, b. Greenwich, N. Y.) Attended Northeastern University, 1931–1935, graduating with degree of B.S. in civil engineering; summers of 1929, 1930, 1931, with Delaware & Hudson Railroad (Delaware & Hudson Student Engineering Corporation); co-operative work with Northeastern, 1931, 1932, engineering department, city of Newton, Mass.; 1934, construction and maintenance department, Colonial Beacon Oil Company; 1935, engineering department, city of Southbridge, Mass.; June, 1935, to present time, construction and maintenance department, Colonial Beacon Oil Company; now clerk, maintenance department, Colonial Beacon Oil Company, Boston, Mass. Refers to *H. B. Alvord, C. O. Baird, C. S. Ell, A. E. Everett, Jr.*

Hollinshead, George Frederick, Jr., Quincy, Mass. (Age 24, b. Braintree, Mass.) Was graduated from Northeastern University, June, 1935, with B.S. in civil engineering; part-time work with Ernest W. Branch, Quincy, Mass., September, 1930, to June, 1934; city of Quincy, P. W. A., June, 1934, to October, 1934, rodman and transitman; Whitman & Howard, Boston, May, 1935, to July, 1935, on race track construction, rodman and transitman; July, 1935, to present time, on E. R. A. and W. P. A. Blackstone Valley project, as transitman and draftsman. Refers to *H. B. Alvord, C. O. Baird, E. W. Branch, A. E. Everett, Jr.*

Jenness, George W., Hanover, Mass. (Age 26, b. Pembroke, Mass.) Entered Northeastern University September, 1929; graduated June, 1934, with degree of B.S. in civil engineering; worked intermittently — about thirty days — summer of 1935, on survey of two hundred acres under W. G. Ford, Marshfield, Mass.; at present civil engineer with R. N. Mayall, deputy district officer, Massachusetts Branch, Historic American Building Survey, 91 Newbury Street, Boston, Mass. Refers to *H. B. Alvord, F. L. Flood, R. N. Mayall, A. Tedesco.*

Knowlton, Kenneth Franklin, Natick, Mass. (Age 22, b. Natick, Mass.) Was graduated 1935 from Northeastern University, degree B.S. in civil engineering (receiver of Desmond FitzGerald Scholarship); co-operative work, July, 1932, rodman, engineering department, city of Newton, Mass.; October, 1933, rodman, Land Court survey, Northeastern University Field, Brookline, Mass.; April, 1934, one week rodman on foundations of new Parcel Post Building, South Station, with C. J. Maney Company; May, 1934, two weeks drafting and tracing work, E. M. Brooks, Newtonville, Mass.; June, 1934, to April, 1935, part-time work transitman on survey work on residential building, Joseph Selyn, Belmont, Mass.; May 14, 1935, to August 24, 1935, rodman on survey parties and drafting and computation work in office of Lewis W. Perkins, North Scituate, Mass.; at present (temporary appointment from August 26, 1935, to November 30, 1935), senior sanitary aid, Massachusetts Department of Public Health, Engineering Division, Boston, Mass. Refers to *H. B. Alvord, W. M. Campbell, F. F. Hampe, F. H. Kingsbury, R. M. Soule.*

ADDITIONS

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Frank J. Marsetta, 121 Mountford Street, Boston, Mass.

Students

Kenneth M. Cline, 37 Howland Street, Roxbury, Mass.

William E. Garlick, 121 East Street, Dedham, Mass.

William H. Stead, 79 Pleasant Street, North Andover, Mass.

Elwin M. Tobin, 7 Melville Place, Lynn, Mass.

DEATHS

Fayette F. Forbes .	Nov. 20, 1935
James W. Rollins .	Nov. 19, 1935

BOOK REVIEW *

"The Work of the Sanitary Engineer," by Arthur J. Martin, Chartered Civil Engineer, M.Inst.C.E., M.Inst.W.E., M.Cons.E., F.G.S., F.R.San.I. Published by MacDonald & Evans, London, 1935.

In this handbook of 488 pages the author has endeavored to describe briefly British practice in sanitary engineering. The book is intended to serve as a guide to candidates coming up to be examined for appointments as sanitary inspectors and health visitors in England. It is the author's hope, also, that the "book will be not without value to the practicing engineer."

The work is divided into six parts dealing, respectively, with administration, water supply, sewerage, sewage disposal, refuse, and flood prevention and land drainage. Technical discussions are necessarily limited. American readers will be particularly interested in the British law relating to sanitation and public health from which the author quotes repeatedly, and in the descriptions of many experiences of the author in his own practice.

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* By Thomas R. Camp, Associate Professor of Sanitary Engineering, Massachusetts Institute of Technology.

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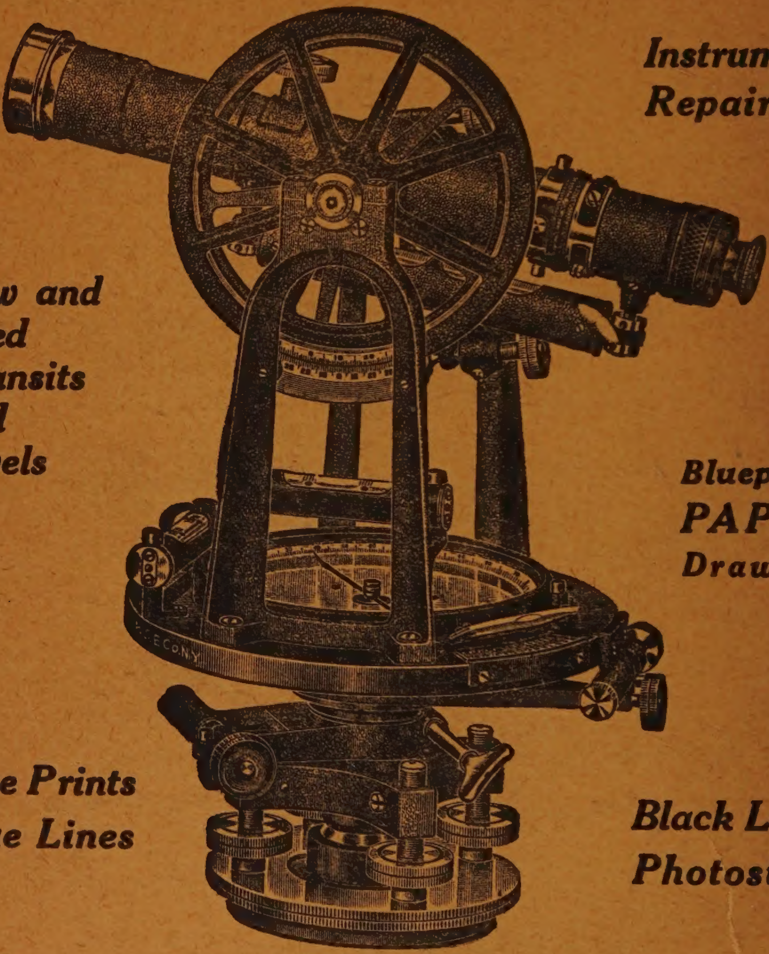

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